SRR-83-07

A Prototype Prestressed Wood Bridge

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Published by:
Research and Development Branch
Ontario Ministry of Transportation and Communications
Hon. James W. Snow, Minister
H.F. Gilbert, Deputy Minister

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The Editor, Technical Publications
Ministry of Transportation and Communications
1201 Wilson Avenue
Downsview, Ontario
Canada M3M 1J8

April 1984

Technical Report Documentation Page

MTC Report Number SRR-83-07	ISBN Number:	Other Document Number:	Date: April 1983
Title: A Prototype Prestressed Wood Bridge			
Authors (name, title and/or organisation): R.J.Taylor, Research Engineer H.Walsh, Regional Engineer, MNR			
Published by: The Research and Development Branch, MTC		Client Group:	
Participating Agencies:			

Abstract:
The transverse prestressing of wood was conceived in 1976 as a method for rehabilitating nailed laminated wood decks. The success of this new concept in rehabilitation resulted in it becoming the subject of a major research and development program conducted by the Ontario Ministry of Transportation and Communications (MTC).
The objective of this paper is to outline the design, construction and load testing of the first new prestressed wood bridge. The bridge was designed by MTC, according to the new design specifications, and constructed by the Ontario Ministry of Natural Resources (MNR) over the West River, on a logging access road, near Espanola, Ontario in 1981.

Key Words: Wood, bridge, prestressed, test design, laminated, OHBDC

Comments:

Distribution: Unrestricted, MTC HO + regions, selected structural and geotechnical engineers

Copyright Status: Crown copyright

TABLE OF CONTENTS

	PAGE
SUMMARY	vii
1/ INTRODUCTION	1
2/ STRUCTURAL SELECTION	2
2.1/ Site Description.	2
2.2/ Structural Description.	2
3/ DESIGN	4
3.1/ Analysis.	4
3.1.1/ Frame Analysis	5
3.1.2/ Orthotropic Analysis	5
3.1.2.1/ Loading/Output	6
3.1.2.2/ Maximum Forces	6
3.2/ OHBDC Specifications.	7
3.2.1/ Maximum Positive Moment	7
3.2.2/ Deck/Leg Connection	9
3.2.3/ Load Distribution	11
3.3/ Summary	12
4/ CONSTRUCTION	13
4.1/ Manufacturing and Treatments.	13
4.2/ Foundations	13
4.3/ Prefabrication.	14
4.4/ Erection and Prestressing	14
5/ LOAD TESTING AND EVALUATION	16
5.1/ Instrumentation	16
5.2/ Vehicle Loading	17
5.3/ Load Testing.	17
5.4/ Evaluation of Load Test Results	17
5.4.1/ Preliminary Review	18
5.4.2/ Movement of Leg Supports	18

PAGE

5.4.3/ Modulus of Elasticity. 19
5.4.4/ Analytical Design Models 20
5.4.4.1/ Orthop 2 21
5.4.4.2/ Orthop 3 22
5.4.4.3/ Summary of Analyses. 24
5.5/ Summary of Load testing and Evaluation. 24

6/ SUMMARY AND CONCLUSIONS 26

7/ ACKNOWLEDGEMENTS 27

REFERENCES 28

APPENDICES
Table A-1 29
Figure A-1 30

LIST OF TABLES

PAGE

1/ Maximum Force Modes Considered in Design. 31
2/ Vehicle Loading 31
3/ Leg Movements Under Eccentric Loading (mm). 32
4/ Leg Movements Under Repeated Eccentric Loading (mm) 32
5/ Shear Modulus and Transverse Modulus of Elasticity of Prestressed Wood Decks [2] 33

LIST OF FIGURES

PAGE

1/ Proposed West River Crossing 34
2/ Elevation Fox Lake Bridge. 34
3/ Galvanized Nail-Plate Connector. 35
4/ Partial Section at Leg 36
5/ Leg Details. 37
6/ Geometry Assumed for Frame Analysis. 38
7/ OHBDC Design Load. 39
8/ Orthotropic Deck Layout. 40
9/ Loading for Orthotropic Analysis 41
10/ Moment INFLUENCE, End Span 42
11/ Test Set-up for Shear Connector. 43
12/ Failure of Shear Connector 44
13/ Load vs Slip of Shear Connector. 44
14/ Test Set-up for Full Leg Connection. 45
15/ Failure of Leg Connection. 45
16/ Cast in Place concrete Support 46
17/ Wood Bearings Anchored to Concrete Support 46
18/ Hydraulic Presses for Splicing 47
19/ Pre-Fabricated Wood Components On-Site 47
20/ Assembled Structure Prior to Prestressing. 48
21/ Hydraulic Prestressing Jack. 48
22/ Leg Support Anchorage. 49
23/ Completed Structure Opened in 1982 50
24/ Linear Displacement Transducer 50
25/ Plan Layout of Deflection Transducers. 51
26/ Additional Instrumentation 51
27/ Demountable Strain Gauge 52
28/ Vehicle Placement. 53
29/ Monitoring Leg Movement. 54
30/ Vehicle Load vs Vertical Displacement at Leg Support 55
31/ Vertical Displacements, Centre Span, Eccentric Loading 56
32/ Vertical Displacements, West Leg Support, Eccentric Loading. . 57

PAGE

33/ Vertical Displacement, West Span, Eccentric Loading. 58
34/ Vertical Displacements, West Leg Support, Eccentric Loading. . 59
35/ Vertical Displacements, West Span, Eccentric Loading 60
36/ Vertical Displacements, West Leg Support, Eccentric Loading. . 61
37/ Vertical Displacements, Centre Span, Concentric Loading. . . . 62
38/ Vertical Displacements, West Leg Support, Concentric Loading . 63
39/ Vertical Displacements, West Span, Concentric Loading. 64
40/ Vertical Displacements, West Leg Support, Concentric Loading . 65
41/ Vertical Displacements, West Span, Concentric Loading. 66
42/ Vertical Displacements, West Leg Support, Concentric Loading . 67
43/ Vertical Displacements, Vehicle Position 3, Eccentric Loading. 68
44/ Vertical Displacements, Vehicle Position 4, Eccentric Loading. 69
45/ Vertical Displacements, Vehicle Position 5, Eccentric Loading. 70
46/ Vertical Displacements, Vehicle Position 3, Concentric Loading 71
47/ Vertical Displacements, Vehicle Position 4, Concentric Loading 72
48/ Vertical Displacements, Vehicle Position 5, Concentric Loading 73
49/ Flexural Strains, Vehicle Position 3, Eccentric Loading. . . . 74
50/ Flexural Strains, Vehicle Position 3, Concentric Loading . . . 75

SUMMARY

The transverse prestressing of wood was conceived in 1976 as a method for rehabilitating nailed laminated wood decks. The success of this new concept in rehabilitation resulted in it becoming the subject of a major research and development program conducted by the Ontario Ministry of Transportation and Communications (MTC). Transverse prestressing of laminated wood decks provides high strength due to load sharing, better load distribution and excellent resistance to the effects of cycle loading. The extensive work performed by MTC over the past seven years has eventually led to the formulation of a set of comprehensive design specifications for prestressed wood.

The objective of the paper is to outline the design, construction and load testing of the first new prestressed wood bridge. The bridge was designed by MTC, according to the new design specifications, and constructed by the Ontario Ministry of Natural Resources (MNR) over the West River, on a logging access road, near Espanola, Ontario, in 1981.

The paper provides a description of the design process, with reference to the new design specifications, which have now been adopted by the Ontario Bridge Design Code (OHBDC). Also included is a description of the computer analysis techniques used to analyse the bridge for design purposes. The OHBDC specifications do not provide for the design of full moment connections as required by this rigid frame structure. Therefore, the report outlines results of a number of full size laboratory tests that were performed on several proposed connections, in order to ensure adequate strength and stiffness.

The field construction process is also outlined, highlighting the prefabrication and assembly of the individual wood laminate frames. The bridge was constructed, for the most part, by the MNR construction crew consisting of three men and a light construction crane. MTC's new multijack hydraulic system, which was specifically assembled for use on prestressed wood bridges, was used to prestress the completed bridge.

In summary, the report describes the load testing of the bridge in 1982 and the subsequent evaluation of the test results. The structural performance indicated that it is an extremely rigid structure with considerable reserve strength.

MNR indicated that the West River Bridge costs only two-thirds the price of the steel bridge originally proposed for the site. Its life expectancy is believed to be in excess of 50 years. It is hoped this paper provides adequate technical information and incentive so that other jurisdictions may seriously consider the use of prestressed wood as an economical alternative for bridge construction.

1/ INTRODUCTION

The transverse prestressing of laminated wood decks was conceived in 1976 [1] as a method of rehabilitating existing nailed decks. The success of this new concept in rehabilitation results in it becoming a major research and development program [2] conducted by the Ontario Ministry of Transportation and Communications (MTC). This extensive research and development work led to the formulation of a comprehensive set of design specifications [3] devoted entirely to the design of prestressed wood decks. These new specifications will be included in the 1983 edition of the Ontario Highway Bridge Design Code (OHBDC) [4]. In order to evaluate the effectiveness of these new specifications, MTC and the Ontario Ministry of Natural Resources (MNR), designed and constructed the first new prestressed wood bridge in 1981.

The objective of this report is to outline the design, construction and load testing of this prototype prestressed wood bridge. The report includes a description of the design analysis performed, but does not detail all the new OHBDC specifications on prestressed wood, as these are extensively presented in Reference 3.

The fabrication and erection procedures are also outlined with particular emphasis on field construction as conducted by MNR's Field Construction Crew. The report also summarized the loading testing and subsequent evaluation of the completed bridge, as performed by MTC in 1982.

2/ STRUCTURAL SELECTION

The main objective of the structural selection was to optimize the use of the prestressed wood concept while minimizing the on-site construction requirements. Second to this, was the utilization of this prototype to demonstrate the design flexibility of the prestressed wood system. As a result of the latter requirement, the structural selection, while being an efficient economic alternative, was not the only feasible solution using prestressed wood.

2.1/ Site Description

The site is located on MNR's Fox Lake logging access road near Espanola, Ontario. Spanning the West River near the southern highway entrance, it is believed this bridge will be subjected to some of Ontario's heaviest commercial loadings.

The site, as displayed in Figure 1, consists of about a 7-m wide fast moving waterway surrounded by large bedrock formations. The site area has little or no loose top surface material and the vicinity offered only limited quantities of good quality fill material.

A typical elevation at the most narrow crossing is depicted in Figure 2 along with the proposed structural form. In order to avoid the costly removal of the surrounding bedrock and limit the fill requirements, the economic structure length would have to be about 13 m.

2.2/ Structural Description

A single span wood deck of 13 m was not considered feasible and so intermediate supports were necessary. In order to satisfy the site requirements, while at the same time demonstrating the flexibility of the prestressed wood system, the wood frame structure shown in Figure 2 was proposed. This structure utilizes inclined legs providing a clear opening of 7.7 m. It maintains the required bridge elevation with minimum foundation requirements. It also takes advantage of two naturally formed rock ledges situated somewhat symmetrically on either side of the waterway.

Both the legs and the deck were to be constructed as an integrated prestressed laminated system. The individual leg laminations were to be pre-spliced to the deck laminations using the galvanized nail-plate connectors shown in Figure 3. The prefabricated frames would then be shipped to the site and assembled, and the entire structure would be prestressed to form a continuous rigid frame structure. The prestressing bars would be accommodated internally by holes predrilled in the wood laminations.

Some additional structural details are given in Figures 4 and 5, and a more detailed description of the construction procedure will be discussed in Section 4.

3/ DESIGN

The OHBDC specifications do not cover some of the areas required for the design of this frame structure, particularly in the area of load distribution. Therefore, a number of computer analyses are presented as well as some laboratory testing of the deck/leg connection.

In order to simplify the design presentation the as-built structural details, as given in Figures 2, 4 and 5 were assumed. The following discussion takes the form of a design evaluation of the as-built structure.

A preliminary evaluation had indicated that utilizing Ontario Red Pine graded as No. 2 and Better would require laminations of 38 mm x 292 mm for the deck and 38 mm x 190 mm for the legs. In addition, only one-half of the deck laminations would require full leg supports in terms of their moment and axial capacity. The latter, was to be achieved by spacing groups of four laminate frames as shown in Figure 4.

The deadline imposed for construction by the MNR resulted in the commencement of the bridge, based only on a preliminary analysis. Due to availability, Dougles Fir was substituted for Red Pine, and so the following analyses and evaluation are based upon Douglas Fir graded as No. 2 and better.

3.1/ Analysis

Due to the structure's low span to depth ratio, it was believed that simplified static analysis would not properly represent the distribution of load in the structure. However, the application of a costly three-dimensional analysis was not considered practical. Instead, two complementing two-dimensional analyses were performed.

The first, a frame analysis [5] represented the structure in elevation, and so did not consider any lateral distribution of load. This provided maximum moments, shears and reactions per line of wheels of the design vehicle.

The second analysis represented the deck in plan as an orthotropic plate [6], simulating the legs as flexible columns. This type of analysis has already proven to be a fairly accurate representation of prestressed wood decks [1] and so was used to derive the necessary distribution criteria needed to produce the design forces.

3.1.1/ Frame Analysis

Figure 6 describes the frame arrangement used to represent the structure for analysis. All the supports were assumed to allow rotation but no translations, except at the ends of decks. Here, the deck was not allowed to move towards the abutment, but allowed to move away. In the field, the deck was to be seated so that the ends would be butted tightly against the abutment, but other than friction there would be little resistance against movement away from the abutment.

The analysis was performed producing influence lines under the effects of a 100 kN wheel load. Subsequently, the influence lines were used to determine the maximum forces under the effects of the OHBDC design vehicle, which is depicted in Figure 7.

This analysis was also used to determine dead load forces and vertical movement of the deck at the leg support under live load. The latter information enabled the determination of a representative support flexibility for use in the subsequent orthotropic analysis, where the leg supports were simulated by flexible columns.

3.1.2/ Orthotropic Analysis

The orthotropic analysis represents the prestressed deck as a two-dimensional plate with flexural and torsional properties which may be different in two directions. The determination of rigidities for the analysis were based upon the idealizations presented in Section 3 of the OHBDC (A.3.2(d) Reference 4). The material properties were also selected from the OHBDC which provides a longitudinal modulus of elasticity of E_L = 9500 MPa (13.11.2 Reference 4) and transverse modulus and shear modulus of $0.05E_L$ and $0.065E_L$ respectively (A.3.2.(b) Reference 4).

Figure 8 describes the geometry and boundary conditions used to represent the structure in the orthotropic analysis. The end supports were represented by line supports and the legs by flexible columns. Due to the limited number of columns that can be given in this program, only one-half of one support was represented in detail. Here, each leg, consisting of four laminations, was simulated by a column with dimensions equal to the horizontal projection of the leg/deck connection. All other columns represented two legs combined.

3.1.2.1/ Loading/Output

The analysis was used to produce influence lines as a means of determining the critical vehicle placement. The unit load was a 100 kN axle as shown in Figure 9, with dimensions equal to the heavier axles of the OHBDC design vehicle. One lateral position of the vehicle was considered with the edge of the outside wheel 0.2 m from the curb as shown in Figure 9.

Output for moments and longitudinal shears was determined to be critical along the centre of the outside line of wheels as indicated in Figure 8. Interlaminate shear or transverse shear was obtained adjacent to the inside line of the wheels.

3.1.2.2/ Maximum Forces

All the force modes considered in the design of the bridge are listed in Table 1. Since the analysis did not properly represent the leg supports the determination of moments and shears in the leg were based upon the following assumptions.

The distribution of load used to determine moment at the top of the leg was assumed to be the same as for negative moment in the deck over the leg, as determined from the orthotropic analysis. The distribution of load used to determine horizontal shear in leg/deck connection was assumed to be the same as for the column reaction determined from the orthotropic analysis. In effect this is the same as projecting the column reaction obtained from the orthotropic analysis, through the leg angle to the horizontal. These distribution widths were then applied to the undistributed leg moments and horizontal shears determined from the frame analysis.

The location of the maximum forces produced using the orthotropic analysis were, for the most part, determined to be at the same sections as for the previous frame analysis and under the same load configurations. Due to the low span/depth ratio of this bridge, load distribution plays a critical part in the design analysis. Figure 10 displays the influence lines produced by both analyses for positive moment in the end span. The influence line from the orthotropic analysis peaks very sharply at the point of reference so that only the 200 kN axle of the design vehicle would be most critical. The influence line from the frame analysis, which does not consider load distribution, peaks less sharply. Here the 200 kN axle of the design vehicle still governs, but the two 140 kN axles produce a moment which is 96% of that maximum. This demonstrates the importance of properly accounting for load distribution in short span structures.

Comparison of some distribution widths produced using the orthotropic analysis with the design widths from the OHBDC will be presented in the following section.

3.2/ OHBDC Specifications

MTC Report SRR-83-03 [3] details the design of prestressed wood bridges using the OHBDC specifications. Except in the case of the deck/leg connection which will be discussed separately, all the other design modes for this bridge followed a similar design process. Therefore, only one of these design modes is presented. Additional information on the OHBDC requirements may be obtained by reference to [3] and [4].

The use of the simplified Methods of Analysis provided by the OHBDC, is also evaluated by comparison with some of the distribution widths derived using the orthotropic analysis.

3.2.1/ Maximum Positive Moment

The following is a sample of the design calculations for maximum positive moment based upon the OHBDC specifications as described in References 3 and 4. All clause references are given in brackets.

FACTORED MOMENT CAPACITY

$Mu = f_{bu} S k_d k_m k_s$ (13.22.6)

ϕ = performance factor = 0.9 (13.4.4)

f_{bu} = specified strength = 10.0 MPa (13.11.2(a))

S = section modulus = $\frac{bd^2}{6}$ = 0.75 $\underline{1000 \text{ mm} \times 292^2 \text{ mm}}$

The maximum moment was assumed to occur at a transverse line of butt joints. Therefore the section modulus considers every fourth lamination to be discontinuous.

S = $10.66 \times 10^6 \text{ mm}^3$

k_d = duration of load effect = 1.0 (13.5.3)

k_m = load sharing effect = 1.5 (13.5.6)

k_s = size effect = 1.0 913.6.2.1)

Therefore Mu = 0.9(10.0) (10.66×10^6) (1.5)

Factored Moment Capacity $\underline{Mu = 144 \times 10^6 \text{ N-mm/m width}}$

TOTAL FACTORED LOAD EFFECT

Maximum Unfactored Live Load Moment M_{LL}

M_{LL} = 64.6 kN-m/m width

Corresponding Dead Load Moments M_{DL}

asphalt M_{DL1} = 2.96 kN-m/m width

wood M_{DL2} = 2.91 kN-m/m width

Load Factors at ULS α = (Table 2.5.1)

Live Load and Dynamic Load Allowance $\alpha_L = 1.4$

Dead Load (Asphalt) $\alpha_A = 1.5$

Dead Load (wood) $\alpha_W = 1.2$

Dynamic Load Allowance DLA

A single axle governs maximum positive moment,

therefore DLA = 0.4 (2.4.3.2.3)

DLA must be adjusted by a factor of 0.7 for wood components (2.4.3.2.10)

Therefore DLA = 0.7(0.4) = 0.28

Total Factored Load Effect M_{TOT}

$$M_{TOT} = \alpha M_{DL1} + \alpha_W M_{DL2} + \alpha_L (M_{LL}) (1 + DLA)$$

$$= 1.5(2.96) + 1.2 (2.91) + 1.4(64.6)(1.28)$$

$$M_{TOT} - 124 \text{ kN-m/m width}$$

The Factored Moment Capacity Mu = 144 kN-m/m width, is 16% greater than the Total Factored Load Effect M_{TOT} = 124 kN-m/m width.

3.2.2/ Deck/Leg Connection

The as-built leg connection is described in Figure 5. Originally, only the galvanized nail plate was to be used. According to the analysis and subsequent evaluation using the OHBDC specifications, the moment and shear capacity of the nail plate connectors was more than adequate at the Ultimate Limit State. In fact, the governing mode at the leg connection was determined to be bearing perpendicular to grain on the underside of the deck. However, concerns were raised regarding the durability of the

nail plates under repeated loadings, particularly with respect to horizontal shear movement which could cause softening of the connection. Therefore the steel "T" shear connector shown in Figure 5 was proposed, to resist horizontal shear. This type of connector, slotted tightly into the wood, has already proven to be resilient to the effects of repeated loads in cycle testing performed at MTC [7] as part of the development of a steel/wood composite system.

Considering the deadline for construction and the uniqueness of this particular bridge, extensive development of the connection detail was not considered practical. However, some ultimate static testing was performed in efforts to establish the mode of failure and the ultimate strength of the proposed connection.

The first test was performed on the steel "T" section alone, as displayed in Figure 11. Two identical connectors were tested together to provide stability of the test setup. The failure mode was one of combined steel deformation and wood crushing as shown in Figure 12. The typical load versus slip response is displayed in Figure 13, indicating linear performance up to about 200 kN. This would present 100 kN/steel connector of which one per leg is used. The total factored horizontal shear from the design analysis is only 48 kN/leg therefore the connector is more than adequate.

In comparison, the total factored bearing resistance of the underside of the deck at the leg support is 180 kN/leg. The required axial force in the leg equivalent to this capacity would be about 208 kN/leg. The 100 kN horizontal shear capacity of the steel "T" connector would be equivalent to about an axial force of 200 kN/leg. Therefore, considering the nail plate connector will offer additional horizontal shear strength, the failure mode of the as-built connection would be expected to be one of bearing caused by the leg, perpendicular to the grain of the wood deck.

To verify this condition, a full size connector test was conducted using a setup as shown in Figure 14. The failure mode was as predicted, in bearing perpendicular to the grain of the wood deck as clearly demonstrated in Figure 15. The load at which the first sign of failure

occurred was 385 kN/leg and the ultimate crushing load was 508 kN/leg. No measureable slip movement of the steel "T" connectors occurred. The 385 kN would represent a horizontal shear capacity of about 190 kN/leg, and 330 kN/leg in bearing, far in excess of that required.

The economics of scaling down the leg connection for more efficient material use, was not considered justified. The nail plate connectors were already as thin as practical and a reduction in the steel "T" would not represent any significant saving due to the small quantity being used. However, the excessive strength of the connection ensures durability and allows, with some degree of confidence, the assumption that the structure will perform as a rigid frame.

3.2.3/ Load Distribution

The application of the Simplified Method of Analysis in the OHBDC specifications to this particular structure, was not considered realistic. The basis of the OHBDC's simplified method assumes considerably longer span bridges, where certain distribution assumption hold true. However, due to the short span of this structure the distribution of load would be critical to the design process, and the use of the OHBDC specifications was expected to render liberal results. In order to evaluate this condition, a comparison of distribution widths for moments was made between the orthotropic analysis and the OHBDC. For the purpose of this comparison the distribution width was interpreted as being the assumed width of the bridge, which when evaluated under one line of wheels, for a specific force mode, would produce the maximum force effect which already accounts for the effects of lateral load distribution.

The orthotropic analysis provided maximum positive live load moments of 64.6 kN-m/m and 61.6 kN-m/m of deck width, for the end span and centre span respectively. The frame analysis provided undistributed maximum moments at the same locations, of 91 kN-m and 74 kN-m/line of wheels of the design vehicle. Therefore the design distribution widths for these two modes would be about:

$$\frac{91.0 \text{ kN-M}}{64.6 \text{ kN-m/m}} = 1.4 \text{ m}$$

and $\frac{74.0 \text{ kN-m}}{61.6 \text{ kN-m/m}} = 1.2 \text{ m}$

Applying the OHBDC specifications, as outlined in Reference 3, the distribution width for both the end span and centre span would be about 1.8 m. This would mean the OHBDC provided a considerably better load distribution lowering the design moment.

The rigid leg supports forming a frame may be a contributing factor in the lower distribution widths for this particular bridge. However, in all very short span bridges, care should be taken to assure that proper attention is given to determining the distribution of load.

3.3/ Summary

The moment capacity determined in the design accounted for the occurrence of butt joints in the deck. In actuality these may be ignored as they have been spliced with nail plates and are not in the highest moment region. This would increase the Total Factored Moment Capacity from 144 kN-m to 192 kN-m. This is over 50% greater than the Total Factored Moment. The same is true for nearly all of the design modes. Due to the substitution of Douglas Fir for the proposed Red Pine, the structure is now considerably overdesigned.

4/ CONSTRUCTION

The fabrication and construction of the bridge was carried out by MNR's Field Construction Crew under the occasional supervision of the MTC design engineer. The crew consisted of three experienced construction people. This was, for a period of several weeks, supplemented with up to three additional men, as available, from other local MNR forces.

4.1/ Manufacturing and Treatments

All steel hardware, including the nail-plate connectors and prestressing materials were hot-dip galvanized for protection. Some of the prestressing bars exhibited non-uniform galvanizing. This resulted in minor difficulties during the installation of the anchorage nuts which were made to match the spiral deformation of the bars.

All the wood materials, including the curbs, posts and bearing caps were precut and predrilled prior to pressure preservative treatment with creosote. Only the holes in the deck, necessary to attach the deck to the supports, and the curbs and posts to the deck, were drilled on site. These areas were surface-treated according to the requirements of the OHBDC specifications.

The deck laminations exhibited poor dimensioning of the holes drilled for the internal prestressing bars. This caused some difficulties in alignment and installation of the prestressing bars at the site.

4.2/ Foundations

The abutments and footings were cast-in-place on sound bedrock as shown in Figure 16. The bridge geometry was such that a minimum of concrete was utilized due to the high cost of transporting pre-mixed concrete to the site.

Both the abutments and footings were anchored to the bedrock using grouted reinforcing tendons. This enables the structure to resist the negative reaction forces and high longitudinal forces which this light frame structure exhibits. Its dead load is very low, and so in contrast to

continuous structures of steel and concrete the negative live load reactions are not overcome by gravity.

Solid sawn 290 mm x 290 mm bearing caps were tied down to the concrete supports as shown n Figure 17. This provided flexibility for the tie down of the laminated deck and legs after prestressing had been performed, otherwise the anchorages would resist the effects of prestressing.

4.3/ Prefabrication

The deck and leg laminations were fabricated at a nearby MTC yard to form the required frame geometry. Figure 18 displays the steel hydraulic presses that were constructed for this purpose utilizing MTC's prestressing jacks.

Care was taken to survey the as-built concrete foundations and verify the exact leg location and angles, as minor adjustments could be introduced at this time.

First, the deck laminations were spliced to form full length laminations equivalent to the overall bridge length. Subsequently, the legs were attached using a marked template to ensure proper location and angle.

Once completed, the individual laminated frames were shipped to the site ready for assembly as shown in Figure 19.

4.4/ Erection and Prestressing

Groups of four laminates were pre-nailed together before installation to ensure the proper distribution of the butt joint splices throughout the deck and provide stability during installation.

The prefabricated deck units were lifted into place using a crane. Due to poor prestressing hole locations, the holes had to be enlarged during installation to ensure access for the prestressing bars. The completed laminated wood frame structure is shown in Figure 20 with the prestressing bars and steel bulkheads installed and ready for transverse prestressing.

Prestressing was performed utilizing MTC's 24-Jack Hydraulic System, which enabled the prestressing of the entire bridge at the same time. The prestressing jacks, as shown in Figure 21, are hollow hydraulic cylinders with heavy steel nose plates which allow access to the anchorage nuts during prestressing.

Prestressing was performed according to the OHBDC specifications which requires at least two additional restressings within a specified period. However, the bridge was anchored down to the supports after only the first stressing as this constitutes the majority of transverse movement that will occur. The deck was anchored to the abutment bearings caps using long steel lag screws with a thin steel cover plate. The legs were secured by a pair of reinforced steel angles which were alternatively secured to the legs and the bearing cap as shown in Figure 22.

The completed bridge, shown in Figure 23, was opened to traffic in the Fall of 1981, and has since been used consistently by the local heavy logging traffic.

5/ LOAD TESTING AND EVALUATION

In 1982, MTC performed a live load test on the bridge as part of the evaluation of its structural performance. The primary objective was to develop a better understanding of it's structural response in order to evaluate the analytical model used in the design. In addition, using MTC's special load testing vehicles, the bridge would be proof-tested to a static load of over two and one-half times the legal load.

5.1/ Instrumentation

Primarily, the bridge was instrumented with linear displacement transducers, similar to that shown in Figure 24. A total of thirty-two transducers were installed, as shown in Figure 25, in order to monitor the vertical displacement contour of the deck under load.

A number of displacement transducers were also used to monitor horizontal movements. One was installed at each of the four corners of the deck, as shown by site numbers 45 through 48 in Figure 26. These monitored horizontal movement of the deck with respect to the abutment.

Three transducers were also installed at the deck/leg connection as shown by site numbers 80 through 82 in Figure 26. These monitored differential horizontal movement of the steel "T" connector with respect to the deck.

Four additional transducers were installed during the test to monitor movement of the legs with respect to bearing caps. These are represented by site numbers 83 through 86 on Figure 26. Further discussion of these latter gauges is given in Section 5.3.

Nine demountable strain gauges, similar to that shown in Figure 27, were installed as shown in Figure 26. These measured extreme fiber flexural strain in the deck immediately adjacent to the displacement transducers at the centre of the bridge.

It should be noted that the site numbers provided in Figures 25 and 26 do not run consecutively for several reasons. These numbers represent the print numbers used by the data acquisition system. A number of the sys-

tems channels were not available for static testing, and some others were not functioning properly. The site numbers were selected so as to produce consecutive numbers for groups of gauges performing the same function.

5.2/ Vehicle Loading

Only one of MTC's load testing vehicles was required to test this short-span, single-lane structure. Utilizing the influence lines produced by the design analysis, a number of vehicle positions were selected as shown in Figure 28. Basically, the vehicle was moved through two transverse positions, representing an extreme eccentric loading and a concentric loading. In each lane, the vehicle stopped at five positions, providing a total of ten different static positions of the vehicle. At each of these static positions, all the instruments were recorded.

5.3/ Load Testing

Load testing of the bridge was conducted without any major difficulties and the maximum loading indicated in Table 2 was successfully applied.

However, during the third load level it became apparent, by visual observation, that the leg supports were not resting evenly on the bearing caps. In fact, some movement was required before the support legs became fully engaged. At this time, four additional displacement transducers were installed at the bottom of the legs, to monitor this movement as shown in Figure 29. Though the movement was only around 3 mm, its importance, as will be discussed in the next section, is reflected in the fact that the overall maximum vertical deflection of the bridge was only 9 mm.

5.4/ Evaluation of Load Test Results

Though all four load levels were completed, subsequent on-site review of the recorded information, indicated a large portion of the data from the fourth load level, was not reproduced on to tape. However, it was not considered necessary to repeat that testing, since the remaining data was considered adequate enough to proceed with an evaluation of the bridges structural response.

5.4.1/ Preliminary Review

A preliminary review of the load test results indicated that the vertical deflection contour and the flexural strains would provide an excellent source for evaluating the analytical model used in design.

The results from the additional instrumentation used to monitor support movement and movement of the leg/deck connection was varied. There was no measureable horizontal movement of the deck with respect to the abutment. In addition, there was no measureable slip movement of the leg shear connector with respect to the deck. However, the four transducers measuring movement at the bottom of the legs did register movement under load. Since this movement would effect the overall deflection contour, and subsequently the distribution of load, the initial evaluation was directed towards this phenomenon.

5.4.2/ Movement of Leg Supports

The transducers at the bottom of the legs were not installed until after the second load level. In addition, only the data from the eccentric loading case was complete for the fourth load level. Therefore, the extent of the test results for these particular gauges was very limited.

Table 3 displays the results of the movements at the bottom of the legs for the eccentric loading, in three vehicle loading positions. The increase in load from load level 3 to load level 4 is about 16%, whereas the average increase in the two gauges which produced maximum output was less than 4%. Though not conclusive, it indicates the structure underwent the same amount of leg settlement under both load levels before the legs were resting upon the bearing caps.

Figure 30 displays some vertical deflections measured at the deck level adjacent to the leg support. The same phenomenon is apparent, indicating the maximum movement is reached at the third load level.

A considerable amount of data evaluation was performed in efforts to further the understanding of the leg movement. However, several related factors made it difficult to reach any more specific correlations than have already been presented.

The magnitude of this mechanical movement was apparently also a function of the method of vehicle application. The information displayed in Table 3 and Figure 30 was for vehicle positions which were believed to be applied in exactly the same manner. However, a number of vehicle positions were repeated out of sequence for load level 3. Instead of performing the concentric loading positions immediately after the eccentric positions, the eccentric positions were repeated again before proceeding. The results are displayed in Table 4. The apparent leg movements are definitely reduced over those of Table 3, indicating some of the movement caused by the initial loading, was retained by the structure. Therefore, moving the vehicle over the bridge in a different position, before actually performing the test run would probably change the results. This causes great difficulty in determining correlation of the test data between different loadings and positions as the test vehicle passed over the bridge after each loading, in no defined position, in order to prepare for a subsequent test run.

In summary, it seems the maximum leg movement occurred under load level 3. At this level, the legs apparently were in reasonably good contact with the bearing caps. In order to further investigate the bridge's structural performance, it was necessary to isolate a number of load positions and evaluate the actual deflection contours using the analytical design model.

5.4.3/ Modulus of Elasticity

Usually, when comparing the deflection contours of experimental data with those produced by an analytical model, the comparison is made using normalized curves. This procedure offsets the error introduced by assuming a value for the modulus of elasticity, but it also assumes the structure behaves elastically. Due to the movement of the leg supports, this structure did not perform elastically. Therefore, it was necessary to consider the measured leg displacements in the analysis and use a realistic value for the modulus of elasticity.

The design and construction of this prototype bridge was considered as an MTC research project and so a sample of 17 full-size deck laminations, form the actual treated material supplied for the bridge, was retrieved

at the time of construction. This enabled the determination of a representative modulus of elasticity for use in the analysis. The laboratory test procedure and results are summarized in the Appendix.

Before discussing the deflection comparisons it is necessary to become more familiar with the variation in the modulus elasticity that was measured. The average modulus of elasticity, which was utilized in the subsequent computer analysis, was about 13 300 MPa. This average ignores the highest and the lowest measured values. However, the highest value used in the averaging was 20 300 MPa and the lowest was 7400 MPa, which still covers a very wide range. The coefficient of variation was about 27%.

In reviewing the subsequent deflection and strain comparisons, this wide variation in material property should be kept in mind. The analytical model assumes uniform properties, where in reality the distribution and magnitude of the material properties is quite varied.

5.4.4/ Analytical Design Models

Initially, the bridge was reanalysed using the same orthotropic analysis as was used in the original design described in Section 3. Only the loading was changed to represent the actual test vehicle used in the field. This analysis was designated as Orthop 1, and the results, as compared to the experimental data, are displayed in Figures 31 through 48.

Figures 31 through 42 display deflection contours at various cross sections, under three vehicle positions for both the eccentric and concentric condition. In all cases, the experimental results are of greater magnitude than those of the Orthop 1 analysis, particularly for the sections at the supports. Though the analysis provides for some elastic shortening of the leg support as shown in Figure 34, it does not account for the magnitude of movement which occurs before the leg contacts the bearing cap. There, the apparent deflections, as demonstrated by the experimental results, are much greater.

Figures 43 through 48 demonstrate the same phenomenon using longitudinal deflection profiles taken in the vicinity of a line of wheels. The Orthop 1 analysis provides for little movement of the leg supports, while experimental data indicates an overall positive displacement of the deck.

5.4.4.1/ Orthop 2

The orthotropic analysis [6] provides the means of introducing prescribed settlements for column supports. Therefore it was possible to simulate the leg movement using the analytical model.

According to the Orthop 1 analysis, as demonstrated in Figure 32, the actual elastic movement of the leg supports, assuming no initial free movement takes place, would be very small. Therefore, it was assumed that the deflection contour at the leg, measured experimentally was due primarily to the mechanical movement of the leg.

The orthotropic analysis was then repeated, introducing prescribed settlements for the columns which were based upon the measured deflections given in the figures. Since only the west leg was fully instrumented, a number of assumptions were made. For the symmetrical loading of vehicle position 3, the east leg movements were assumed to be the same as the west leg.

For the other two vehicle positions, the settlement in the east leg was assumed to be zero. This latter determination was based upon the results of the two gauges at the bottom of the east leg, which were less than 0.5 mm or those two vehicle positions.

This analysis was designated as Orthop 2, and the results are also displayed in the appropriate figures. It was not necessary to show the results of Orthop 2 on the figures representing the sections at the support legs, as the analysis now provided support movements nearly identical to the experimental data.

The Orthop 2 analysis, particularly for vehicle position 3, displayed in Figures 31 and 37, provided deflections which were surprisingly close to the experimental deflection contours. The major discrepancy in Figure 31

being at the outside edge of the deck. However the Orthop 2 analysis did not consider the stiffness of the curb, guardrail, and the steel prestressing bulkhead.

In the other comparisons, for vehicle positions 4 and 5, the experimental results are still greater, but not by the same margin. However, the assumption of zero settlement for the east leg was not a fair representation. In fact, the Orthop 2 analysis, by oversight, did not even allow the original elastic movement considered by Orthop 1. It would be possible to arbitrarily introduce settlements which would bring about a better comparison, but this would not be justified.

It is believed that only for vehicle position 3, where the load is longitudinally symmetrical, and east and west leg movements can be assumed to be the same, is the Orthop 2 analysis most realistic. This is clearly demonstrated by the results displayed in Figures 31 and 37.

5.4.4.2/ Orthop 3

One additional, orthotropic analysis was performed which is designated as Orthop 3 on the appropriate figures. It was performed primarily to demonstrate the sensitivity of the structural performance to variations in the material property assumptions.

This analysis reduced the shear modulus (G_{LT}) and transverse modulus of elasticity (E_T) of the deck by a considerable margin. The OHBDC specifies G_{LT} at $0.065E_L$, where E_L is the longitudinal modulus of elasticity, and E_T at $0.05E_L$. These values were used in the Orthop 1 and Orthop 2 analyses.

A recent investigation by Queen's University in Kingston, Ontario [2] indicated that these structural properties for prestressed wood decks are much lower. The report indicated, that for the three species of material considered (Red Pine, White Pine and Hemlock) the G_{LT} and E_T values were functions of the level of prestress and ranges in magnitude as given in Table 5. The last column at 500 kPa, represents the measured prestress level at the test site.

Since Douglas Fir is considered to be a good structural material the

initial assumption was to take the highest values for G_{LT} and E_T at $0.055E_L$ and $0.037E_L$ respectively. These values were used in the Orthop 3 analysis.

In addition to the property changes, the elastic movement for the east leg support was reintroduced for vehicle positions 4 and 5. This would allow at least some positive or negative movement of the leg. Along with this, the stiffness of the wood curb was also introduced as an edge beam, in the attempt to better represent the real edge conditions.

The results of the Orthop 3 analysis are also presented in the appropriate figures, and though the changes are not too dramatic they do demonstrate a trend. The reduction in torsional and transverse stiffness has caused the transverse curvatures to become more pronounced. The new deflected shapes are closer to the experimental results particularly in Figure 31 and 37.

The added freedom of the east support had little effect on the deflected shapes but the increase in edge stiffness did reduce the edge deflections bringing them closer to the experimental results. If the edge stiffness of the steel channel prestressing bulkhead could also be accommodated, the representation would be better.

In addition to the vertical displacement comparisons, several strain comparisons were also made with the analytical models. The transverse line of demountable strain gauges, used at the centre of the bridge, provided a contour of the flexural strain on the underside of the deck.

Figure 49 and 50 compare the experimental strains with those derived from the moments produced by the analytical models. This derivation utilized the average experimental modulus of elasticity and assumed a uniform deck thickness.

Considering the variation in the modulus of elasticity that exists between the actual laminations, the comparison between analytical and experimental results is considered as fair. The applicability of these

demountable gauges to wood, was still under investigation and this particular application was considered a part of that investigation..

Figures 49 and 50 also demonstrate a very important consideration with respect to the three analytical models. The flexural strain distributions directly reflect the moments distributions. According to the figures, these distributions changed very little, even from the original design model to that where the leg movements were introduced in Orthop 1 and Orthop 3. Even the magnitude did not change appreciably considering the positive moment at the centre of the bridge should increase when the intermediate supports undergo settlement. The increase in maximum positive moment in the centre span was only 15% from the original analytical design model, Orthop 1, to the final model, Orthop 3 as shown in Figure 49.

5.4.4.3/ Summary of Analyses

The use of the orthotropic analysis to represent the structural response of a represtressed wood deck system appears to be fairly accurate in terms of displacement contours. The use of lower torsional and transverse flexural stiffness should be considered in design though, apparently, they have limited effect on the distribution.

With regard to this particular bridge, the leg movements do not appear to affect the force distributions appreciably. According to the analytical models, only the maximum positive moment in the centre span is increased by the settlement of the leg supports. All other design force modes were reduced. This 15% increase in the positive design moment is comfortably accommodated by the moment capacity which was determined to be 16% larger than the original factored moment determined in Section 3.

Further modification of the analytical model was not considered justified as small changes in the parameters do not appear to change the structural response to any appreciable degree.

5.5/ Summary of Load testing and Evaluation

The prototype bridge successfully carried MTC's maximum load testing vehicle which represents over two and one-half times the legal load.

Under this load, the maximum vertical displacement of the deck was around 9 mm.

The structural performance of the bridge was found to be representative, using an orthotropic plate analysis, simulating the leg supports as flexible columns. The analytical model indicated the leg movements have no adverse effects on the capacity of this structure to carry the full design load.

6/ SUMMARY AND CONCLUSIONS

The design and construction of this prototype wood bridge demonstrated the flexibility in application of the prestressed wood system. The resulting structure is a strong and durable rigid frame which has an anticipated life expectancy of over 50 years.

The construction of the bridge was conducted by what may be considered semi-skilled labour in terms of bridge construction. Few problems were encountered in the construction apart from that caused by the prestress hole dimensioning which was the fault of the manufacturer. Subsequent to the construction of this bridge, the Sioux Narrows Bridge in Northwestern Ontario was rehabilitated with a new prestressed wood deck in 1982. This new deck consisted of a 120 m long continuous prestressed longitudinally laminated deck. The practicality of utilizing internal prestressing was demonstrated here, as no problems were encountered in assembly and alignment of the prestressing holes in this very long deck.

Very little special equipment was required for the construction of the prototype, apart from MTC's hydraulic prestressing system and a light construction crane. A minimum of on-site work was required as the majority of the fabrication was performed before transportation to the site.

The MNR estimated the cost of this prototype bridge to be only two-thirds the cost of the originally proposed steel structure. According to the load testing and subsequent analytical evaluation, the capacity of the bridge is more than adequate to carry the full OHBDC design loads.

It is believed the prestressed wood system can now be considered as an economical alternative for new-short span bridges [2] as well as a method for rehabilitation [1]. The system is currently being evaluated for use as a transverse laminated wood deck on steel girder bridges, in replacement of the old nailed deck system. Two prototypes have been designed and are expected to be implemented in the Fall and Winter of 1983.

Additional information on the research and development or design of prestressed wood may be found in the references. The work on the 120 m long prestressed deck replacement at the Sioux Narrows bridge is expected to be published in an MTC Structural Research Report by the end of 1983.

7/ ACKNOWLEDGEMENTS

The authors wish to express their appreciation to Mr P.F. Csagoly who conceived the original rigid frame concept for this site using the prestressed wood system. As Principal Research Officer for MTC until 1981, Mr Csagoly was also the major driving force behind the successful development of the transverse prestressing system.

The contributions of the following persons and/or organizations to the successful construction and testing of this new prototype wood bridge are also acknowledged:

S. Cross, Foreman and the Huntsville Field Construction Crew, MNR, Huntsville, Ontario.
J. Klubal, Senior Structural Research Technician, MTC, Toronto, Ontario.
B. Kwok, Senior Laboratory Research Technician, MTC, Toronto, Ontario
L. Pena, Laboratory Research Technician, MTC, Toronto, Ontario.
G. Giles, Laboratory Research Technician, MTC, Toronto, Ontario.
A. Messervia, Foreman, Thunder Bay District Bridge Crew, MTC, Thunder Bay, Ontario.

REFERENCES

[1] Taylor, R.J., and Csagoly, P.F., "Transverse Post-Tensioning of Longitudinally Laminated Timber Bridge Decks," Research Report RR. 220, MTC., Downsview, Ontario, June 1979.

[2] Taylor, R.J., Batchelor, B. deV., and Van Dalen, K. "Prestressed Wood Bridges". International Conference on Short and Medium Span Bridges, CSCE, Toronto, 1982. Reprinted as MTC Structural Research Report, SRR-83-01.

[3] Taylor, R.J., "Design of Prestressed Wood Bridges Using the Ontario Highway Bridge Design Code." MTC Structural Research Report SRR-83-03, Toronto, Revised March 1984.

[4] "Ontario Highway Bridge design Code," Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

[5] Ayres, R.K., "Two Dimensional Frame Analysis Program," Unpublished Development as an MTC Research Project, Toronto, 1975.

[6] Bakht, B., and Bullen, R.C., HECB/B/1, ORTHOP, Analysis of Orthotropic Right Bridge Decks,"(Version 2), Highway Engineering Computer Branch, Department of the Environment, London, England.

[7] Csagoly, P.F., and Taylor, R.J. "A Development Program for Wood Highway Bridges," Structural Research Report, 79-SRR-7, MTC., Downsview, Ontario, 1979.

Appendix

Specimen No.	Calculated Modulus of Elasticity (MPa)
1	9000
2	14310
3	9780
4	Lowest (6980)
5	15530
6	11620
7	15690
8	9380
9	12070
10	Highest (35 300)
11	17590
12	15920
13	15610
14	10500
15	20270
16	14570
17	7420

Standard Deviation σ

$$\sigma = \sqrt{\frac{\Sigma (x - \bar{x})^2}{(n - 1)}}$$

where $\bar{x}$ = mean = 13300 MPa
and number of samples
n = 15
σ = 3640 MPa

Coefficinet of Variation

$$C.V. = \frac{\sigma}{\bar{x}} \times 100\%$$

C.V. = 27.4%

Table A.1/ Averaged Modulus of Elasticities

Appendix

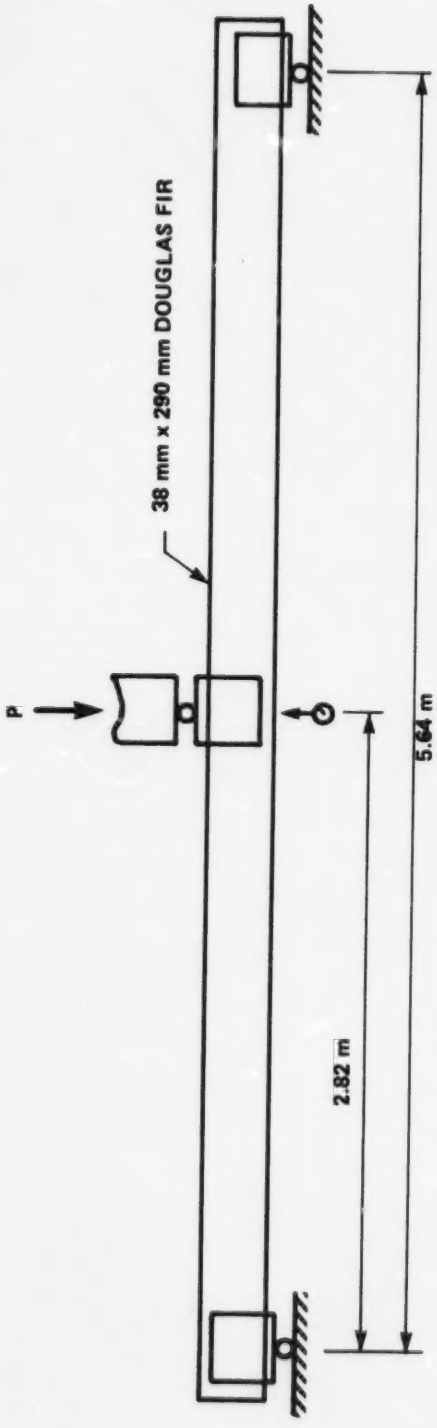


Figure A.1/ Test Set-up for Elastic Testing

DECK	Moment*, Vertical Shear, Interlaminate Shear, Axial Force* & Bearing
LEG	Moment* & Axial Force*
LEG/DECK CONNECTION	Moment*, Horizontal Shear*, & Bearing

* Evaluated as combined effects

Table 1/ Maximum Force Modes Considered in Design

Load Level	Axle Weights			Gross Vehicle Weight
	Steering Axle kg	Front Tandem Axle kg	Rear Tandem Axle kg	kg
1	5.3	25.5	25.7	56.5
2	5.6	31.3	31.1	68.0
3	5.9	37.1	36.5	79.5
4	6.1	42.5	42.3	90.9

Table 2/ Vehicle Loading

Site Number	Load Level 3 Load Positions 2	3	4	Load Level 4 Load Positions 2	3	4
83	2.4	2.3	2.3	2.5	2.5	2.6
84	0.2	0.2	0.3	0.2	0.3	0.4
85	0.3	0.3	0.0	0.4	0.4	0.1
86	3.0	2.8	0.6	3.1	3.0	0.5

Table 3/ Leg Movements Under Eccentric Loading (mm)

Site Number	Load Level 3 (Repeated) Load Positions 2	3	4
83	2.0	2.0	2.1
84	0.2	0.2	0.3
85	0.3	0.3	0.0
86	2.5	2.4	0.3

Table 4/ Leg Movements Under Repeated Eccentric Loading (mm)

Prestress Pressure	275 kPa		900 kPa		Interpolated at 500 kPa	
Material Property	G_{LT}	E_T	G_{LT}	E_T	G_{LT}	E_T
Hem-Fir	$0.03E_L$	$0.015E_L$	$0.04E_L$	$0.03E_L$	$0.034E_L$	$0.02E_L$
Red Pine	$0.05E_L$	$0.03E_L$	$0.065E_L$	$0.05E_L$	$0.055E_L$	$0.037E_L$
White Pine	$0.045E_L$	$0.02E_L$	$0.07E_L$	$0.04E_L$	$0.054E_L$	$0.027E_L$

Table 5/ Shear Modulus and Transverse Modulus of Elasticity of Prestressed Wood Decks [2]

Figure 1/ Proposed West River Crossing

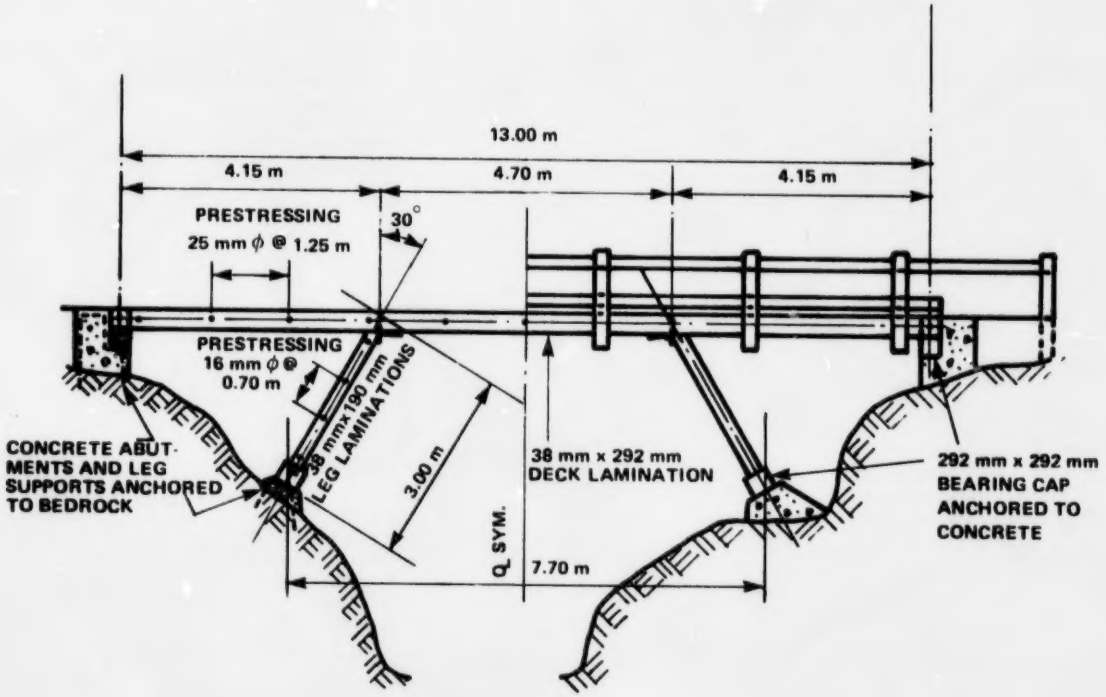


Figure 2/ Elevation Fox Lake Road Bridge

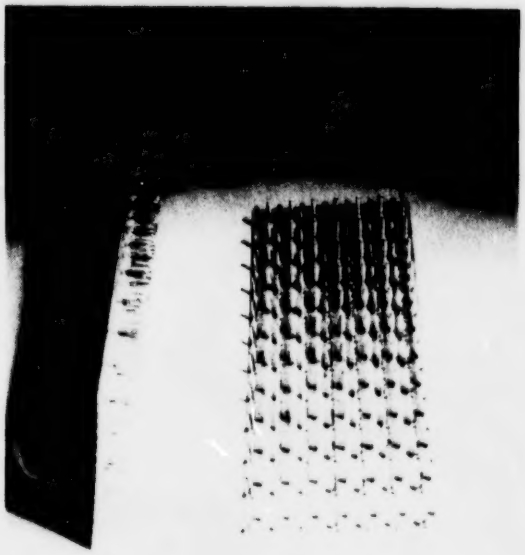


Figure 3/ Galvanized Nail-Plate Connector

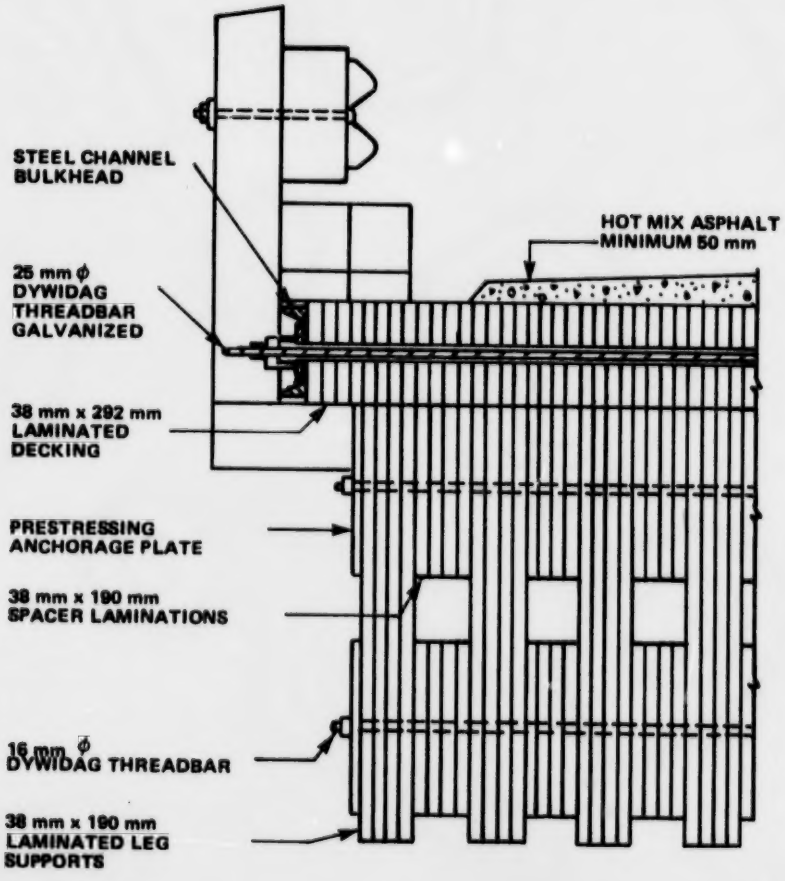
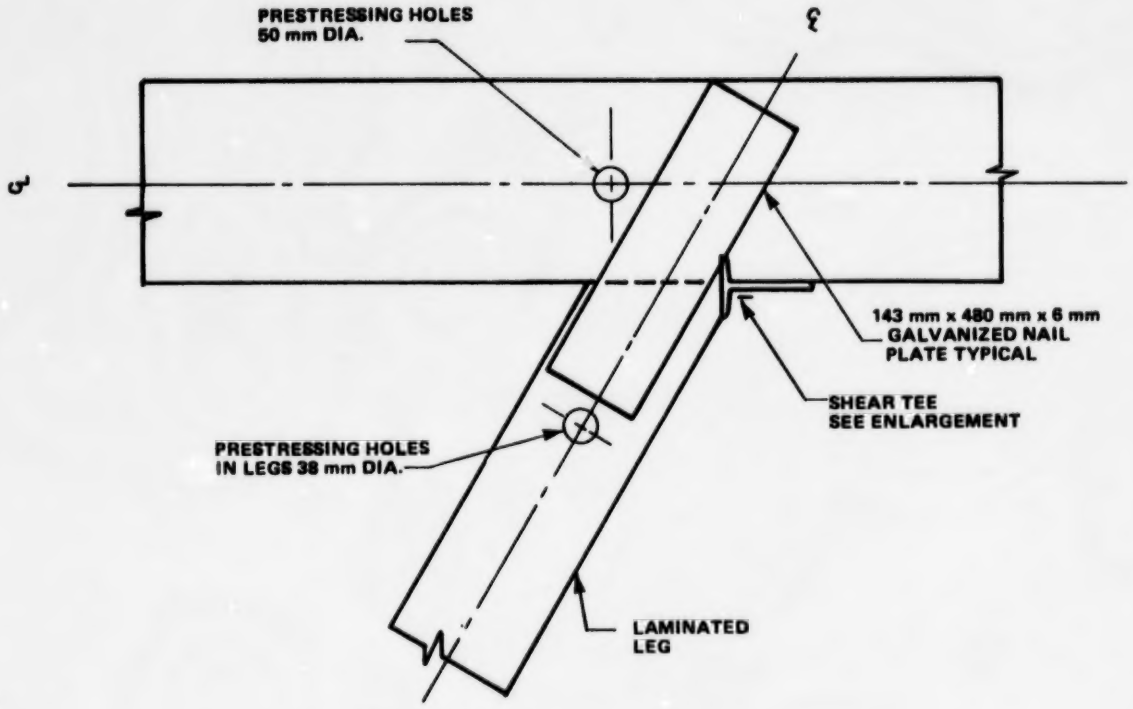
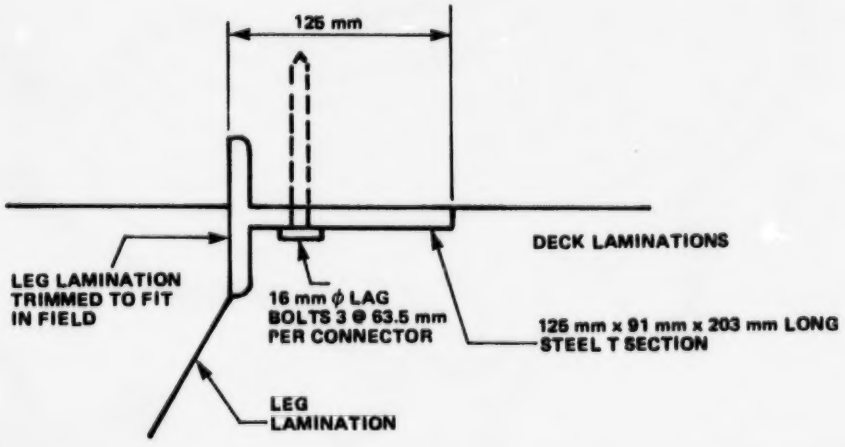


Figure 4/ Partial Section at Leg



LEG CONNECTION DETAIL



T CONNECTOR DETAIL (INSTALLED IN FIELD)

Figure 5/ Leg Details

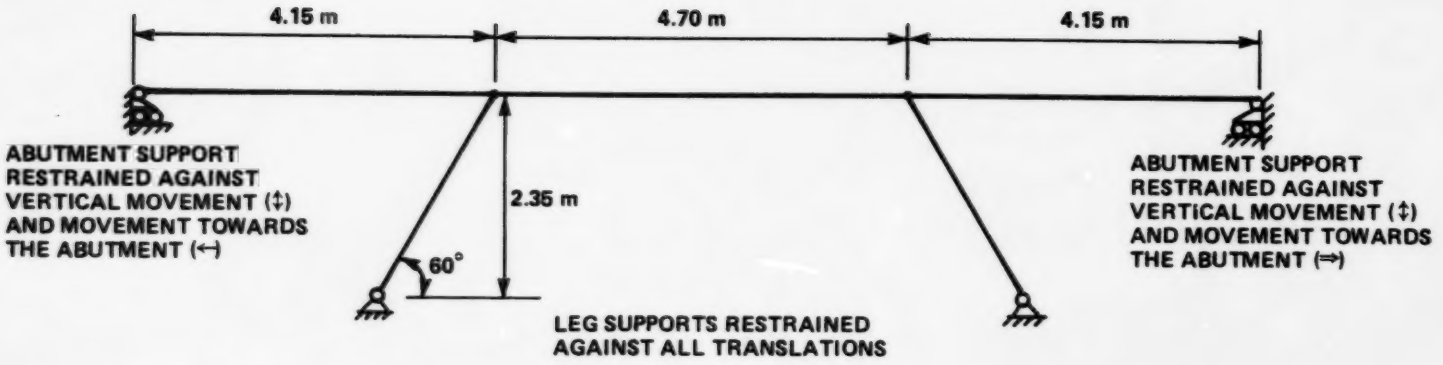

4.15 m
4.70 m
4.15 m
ABUTMENT SUPPORT RESTRAINED AGAINST VERTICAL MOVEMENT (↕) AND MOVEMENT TOWARDS THE ABUTMENT (↔)
2.35 m
60°
ABUTMENT SUPPORT RESTRAINED AGAINST VERTICAL MOVEMENT (↕) AND MOVEMENT TOWARDS THE ABUTMENT (⇔)
LEG SUPPORTS RESTRAINED AGAINST ALL TRANSLATIONS

Figure 6/ Geometry Assumed for Frame Analysis

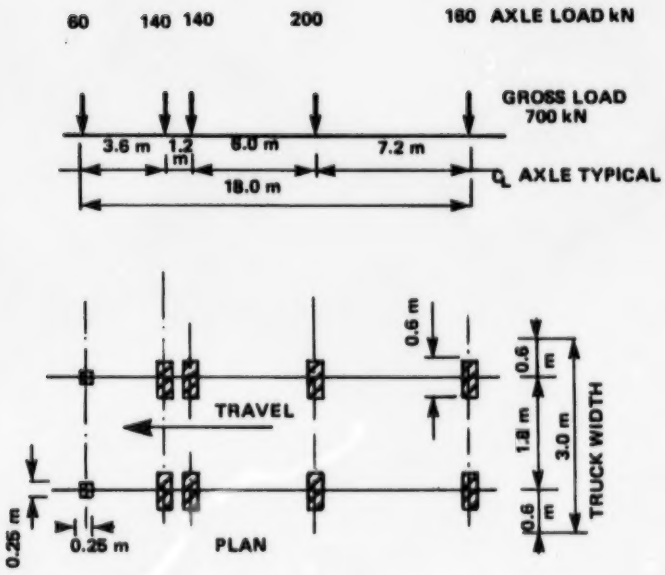


Figure 7/ OHBDC Design Load

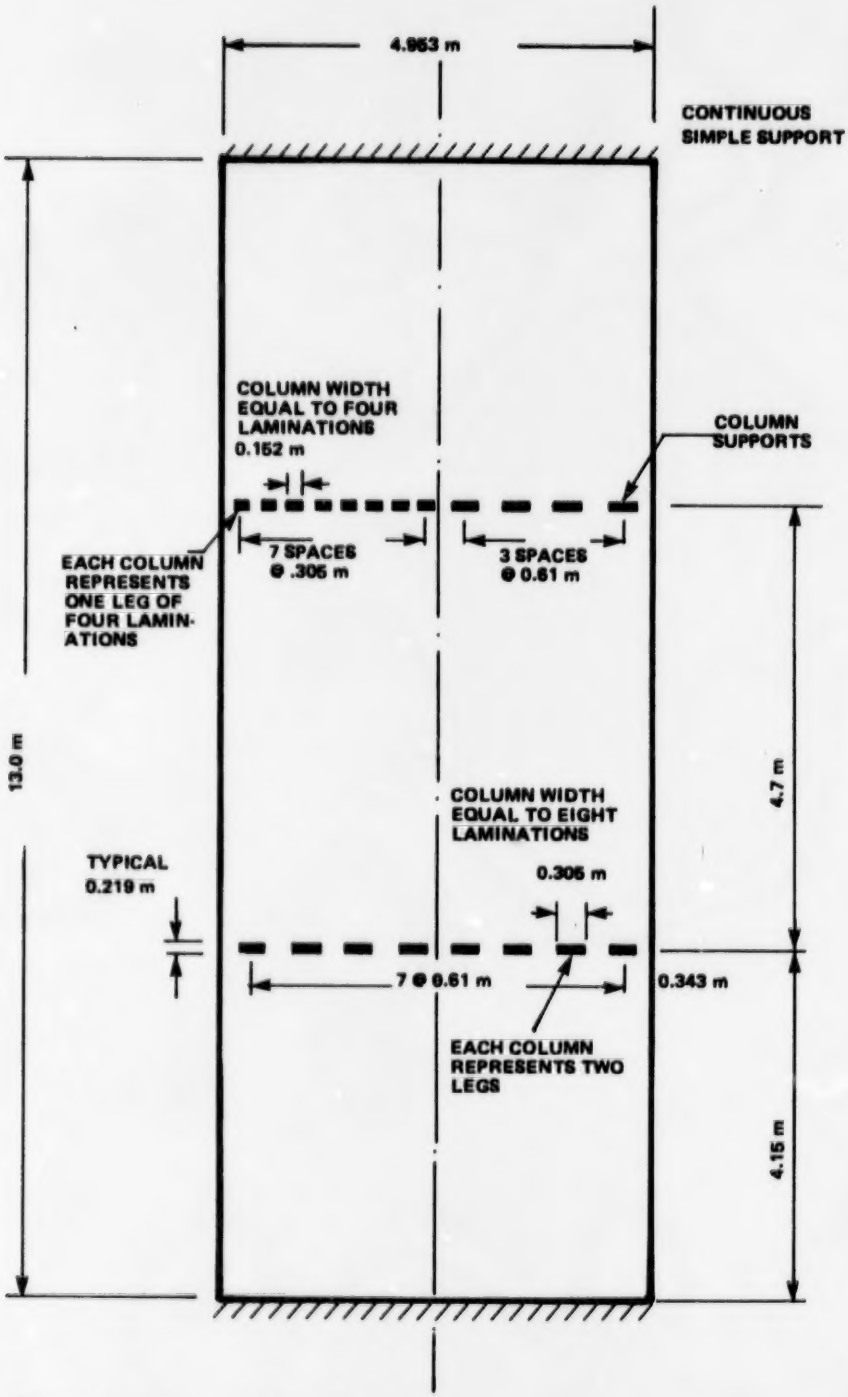


Figure 8/ Orthotropic Deck Layout

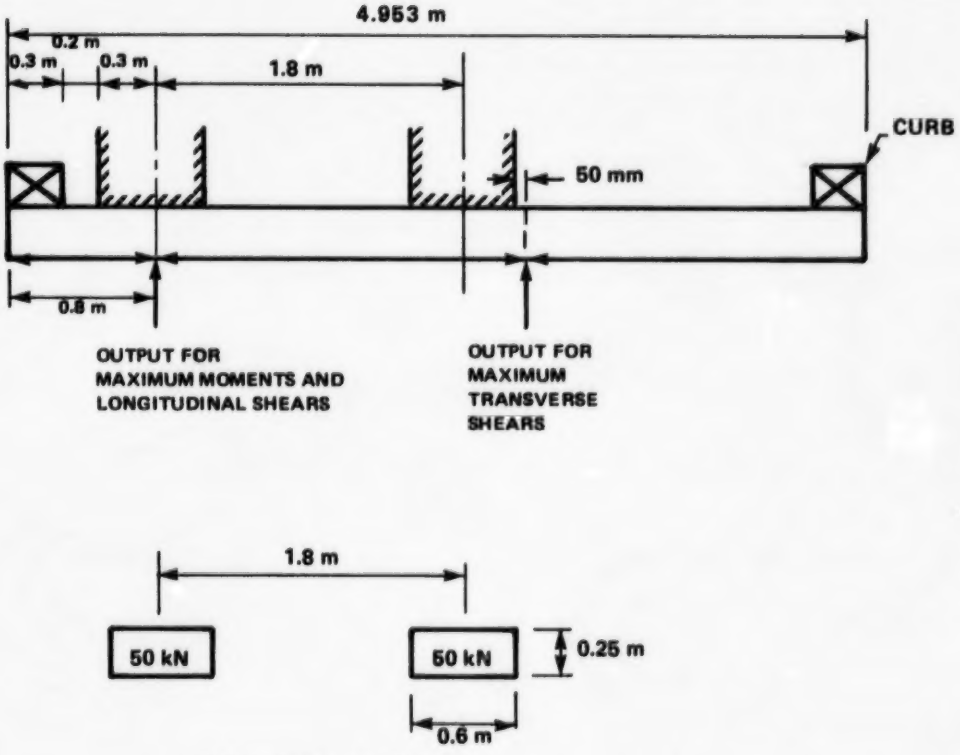


Figure 9/ Loading for Orthotropic Analysis

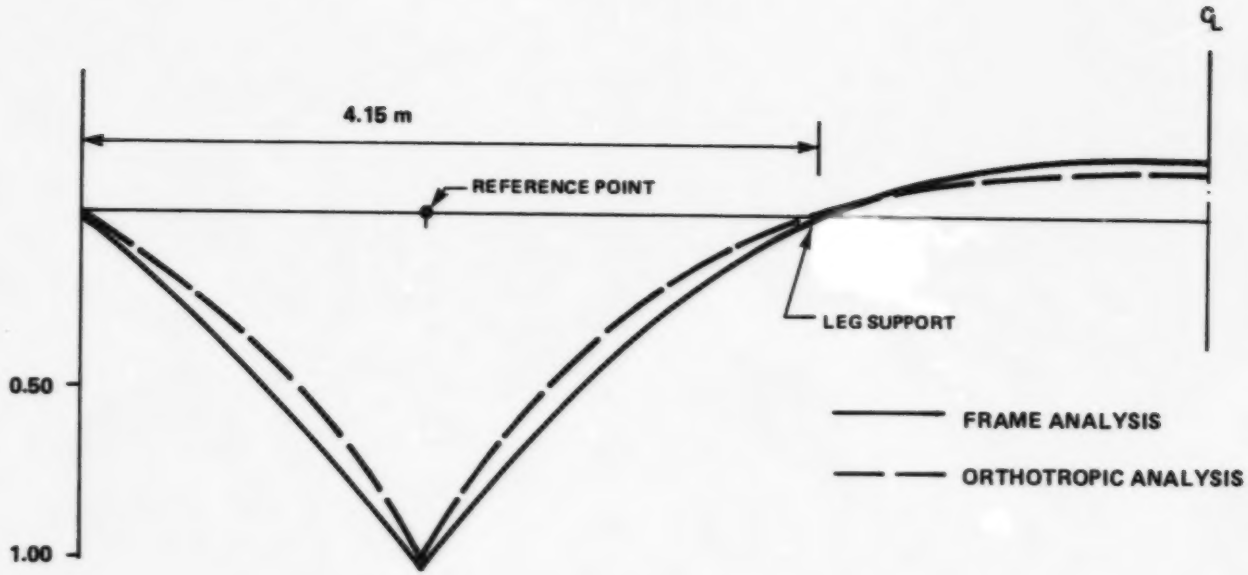
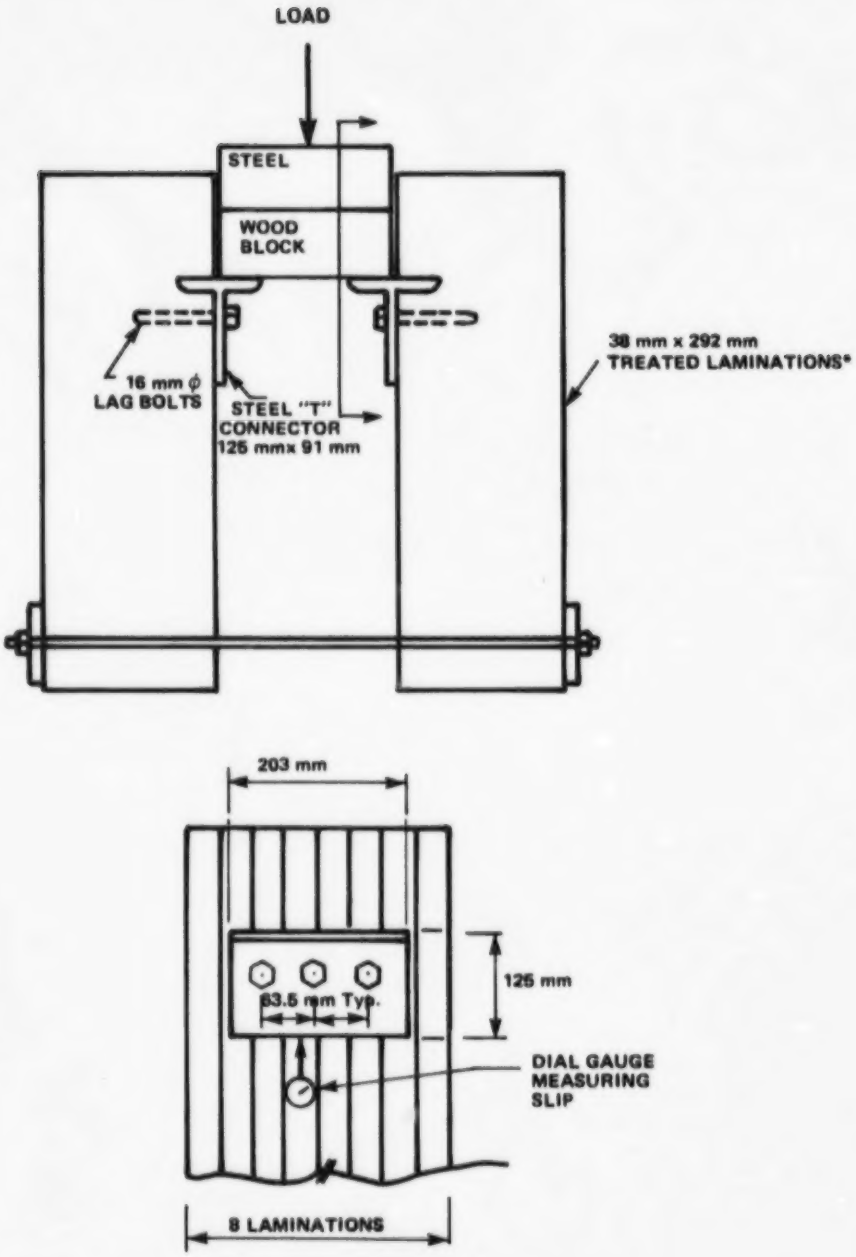


Figure 10/ Moment Influence, End Span



* Test Material Obtained from Actual Bridge Supply

Figure 11/ Test Set-up for Shear Connector

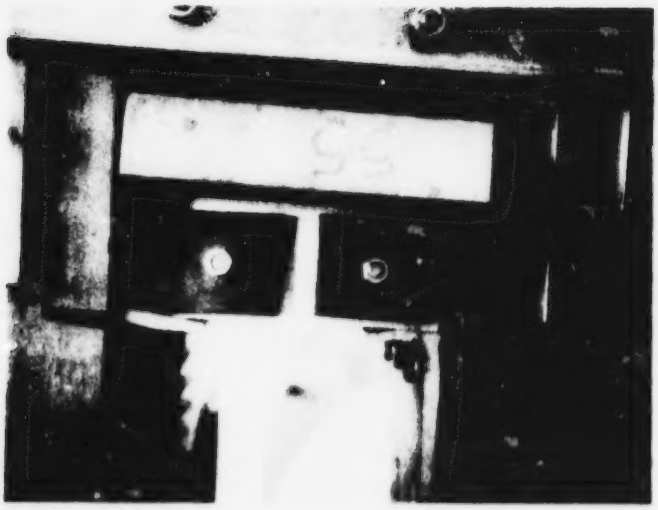


Figure 12/ Failure of Shear Connector

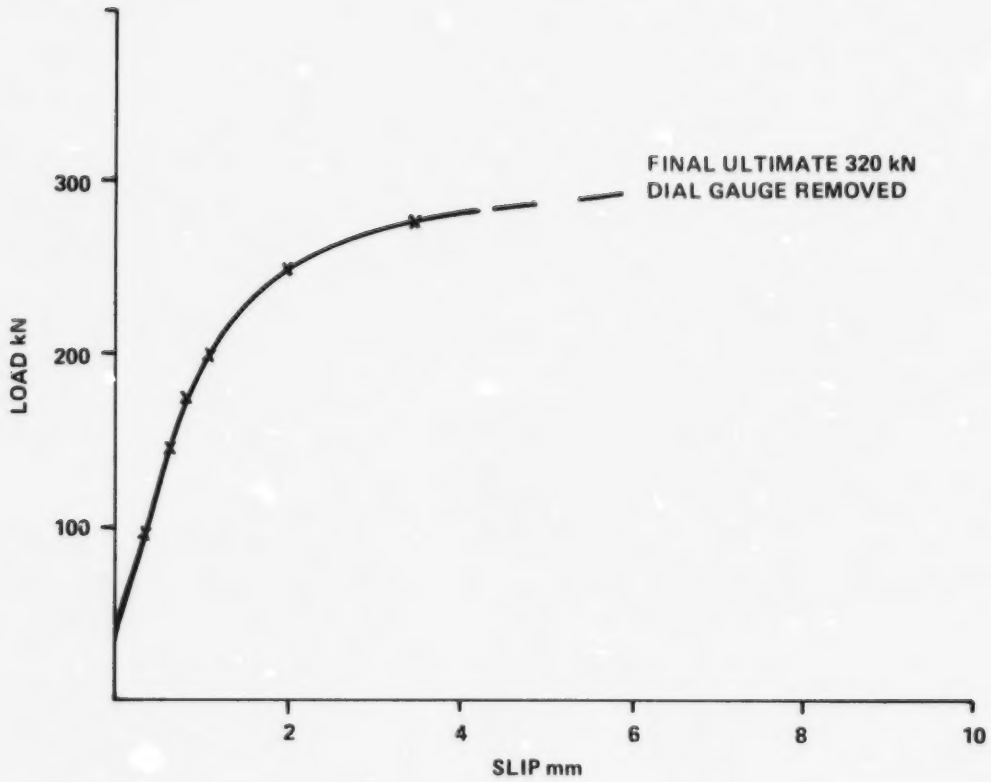
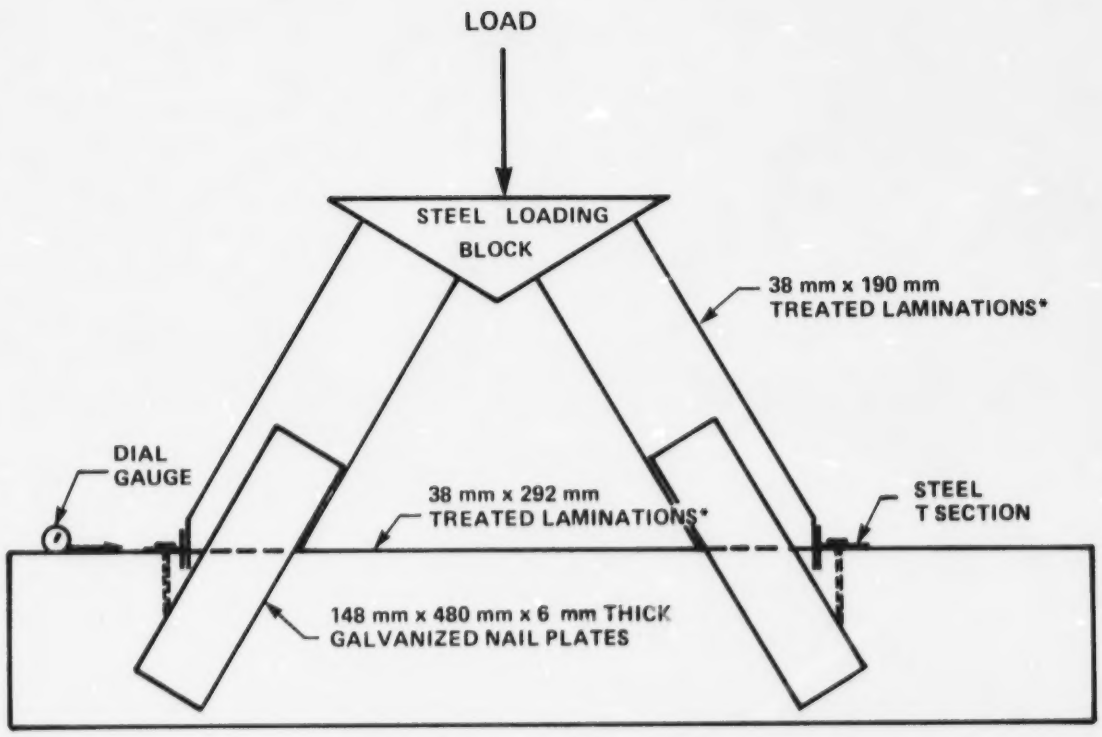


Figure 13/ Load vs. Slip of Shear Connector


LOAD
STEEL LOADING BLOCK
38 mm x 190 mm TREATED LAMINATIONS*
DIAL GAUGE
38 mm x 292 mm TREATED LAMINATIONS*
STEEL T SECTION
148 mm x 480 mm x 6 mm THICK GALVANIZED NAIL PLATES

* Test Material obtained from actual bridge supply

Figure 14/ Test Set-up for Full Leg Connection

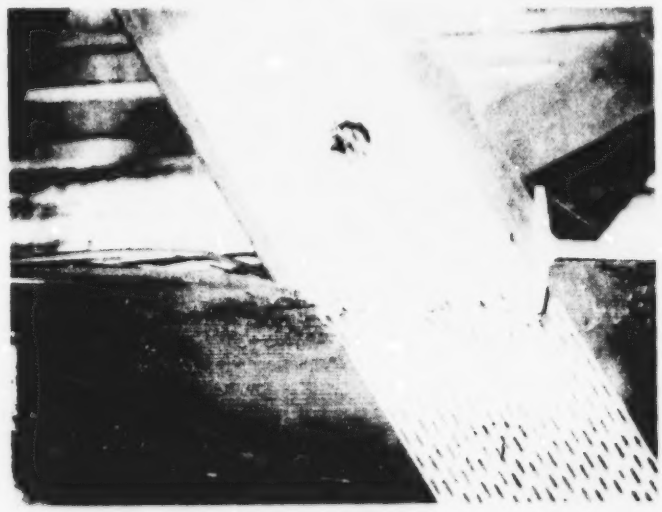


Figure 15/ Failure of Leg Connection

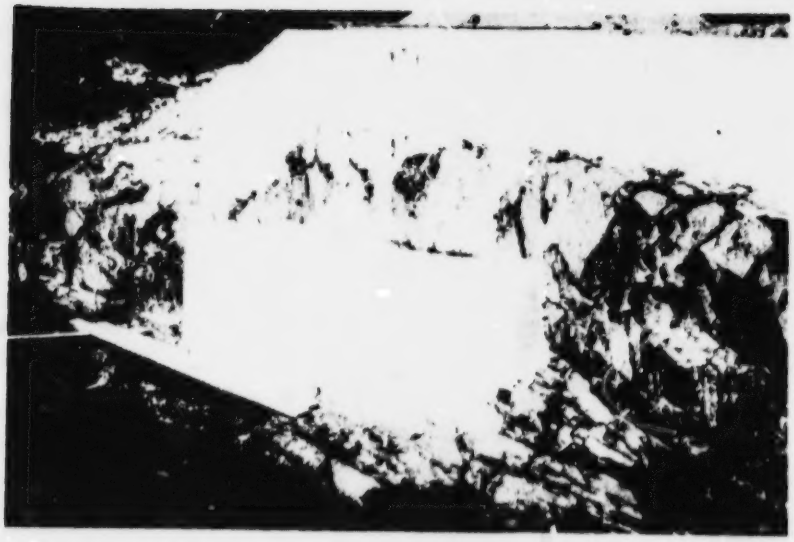


Figure 16 / Cast in Place Concrete Support

Figure 17/ Wood Bearings Anchored to Concrete Support

Figure 18/ Hydraulic Presses for Splicing

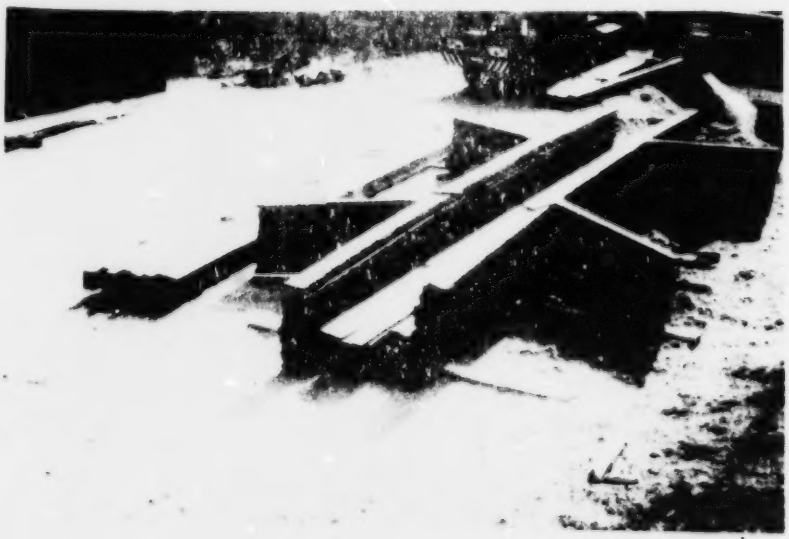


Figure 19/ Pre-Fabricated Wood Components On-Site

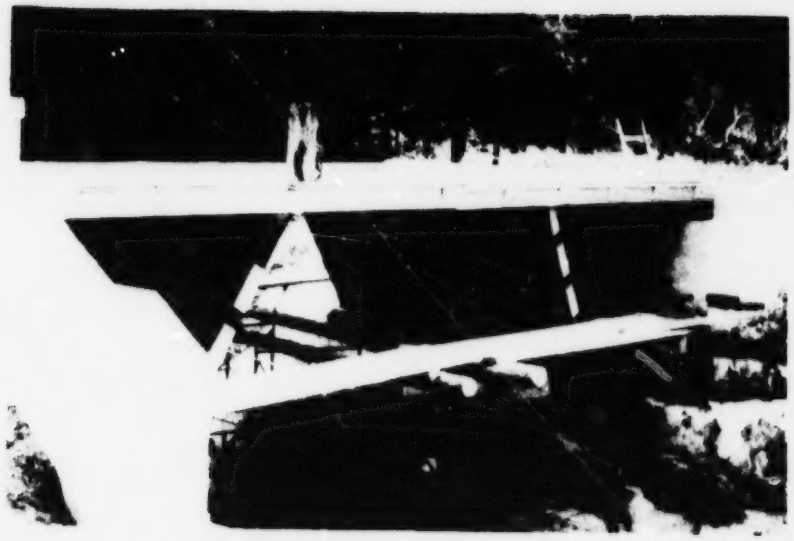


Figure 20/ Assembled Structure Prior to Presrressing

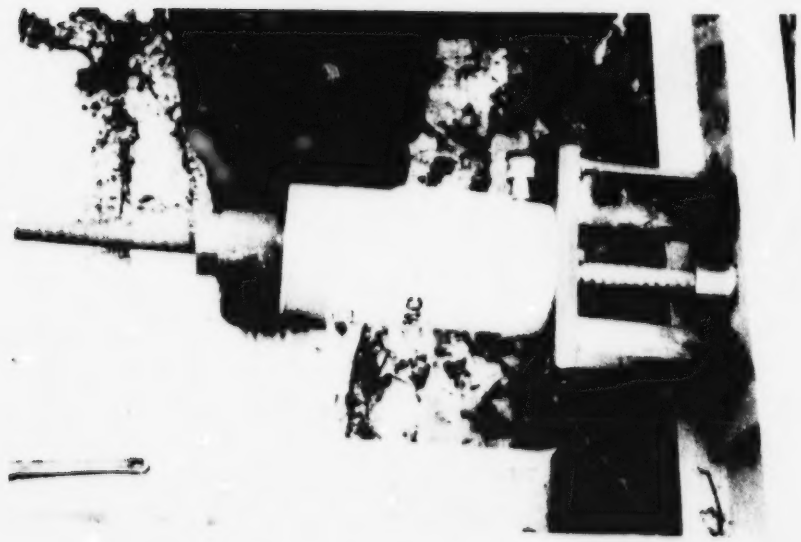
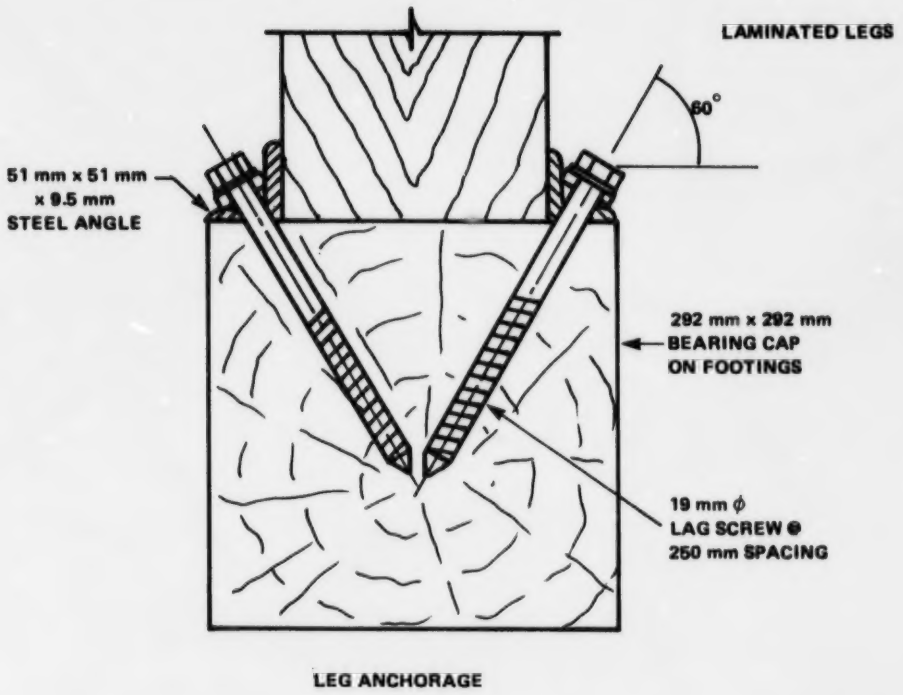


Figure 21/ Hydraulic Prestressing Jack



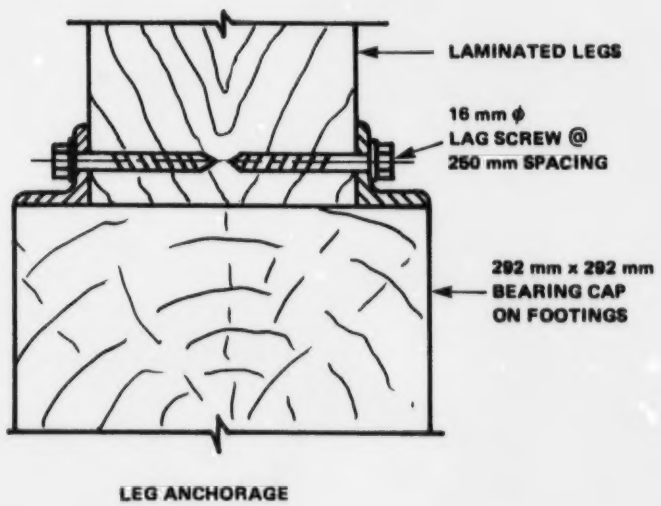


Figure 22/ Leg Support Anchorage

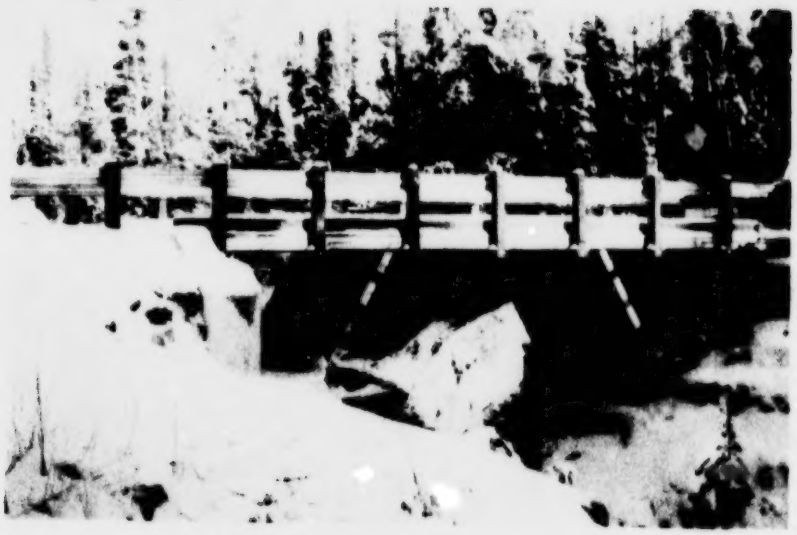


Figure 23/ Completed Structure Opened in 1982

Figure 24/ Linear Diaplacement Transducer

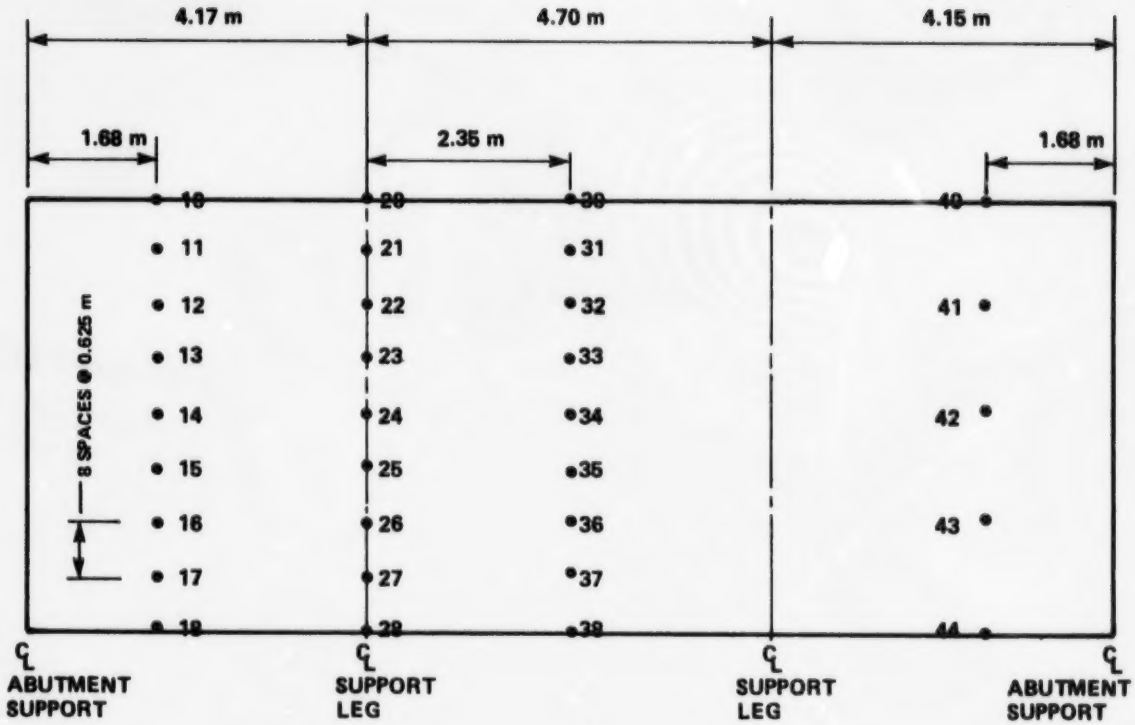


Figure 25/ Plan Layout of Deflection Transducers

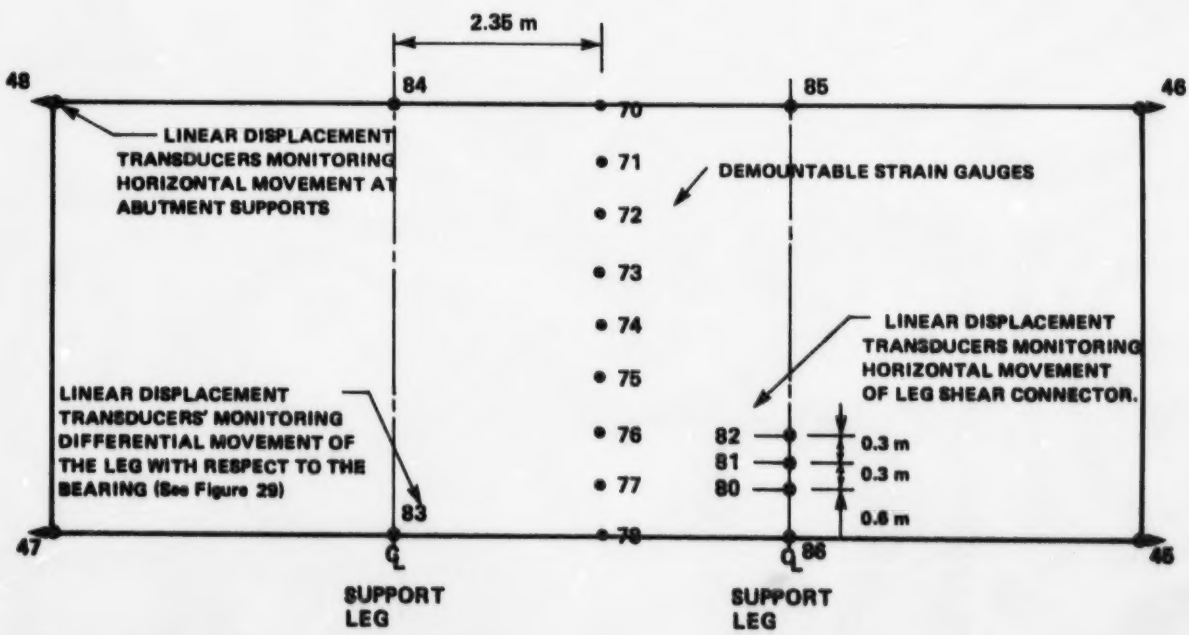
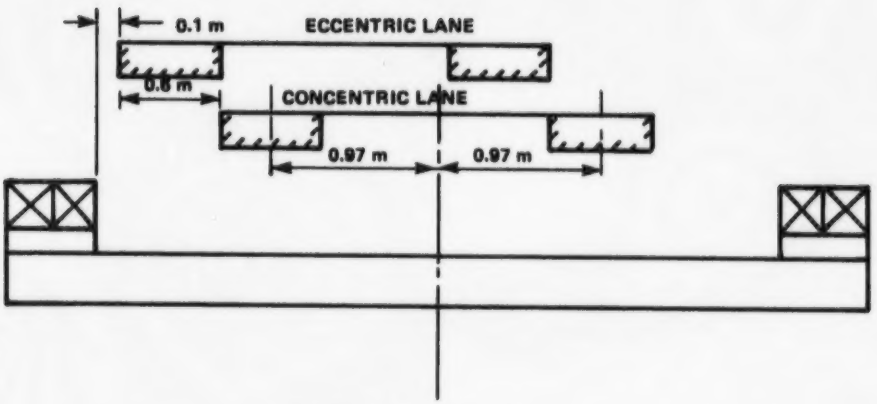
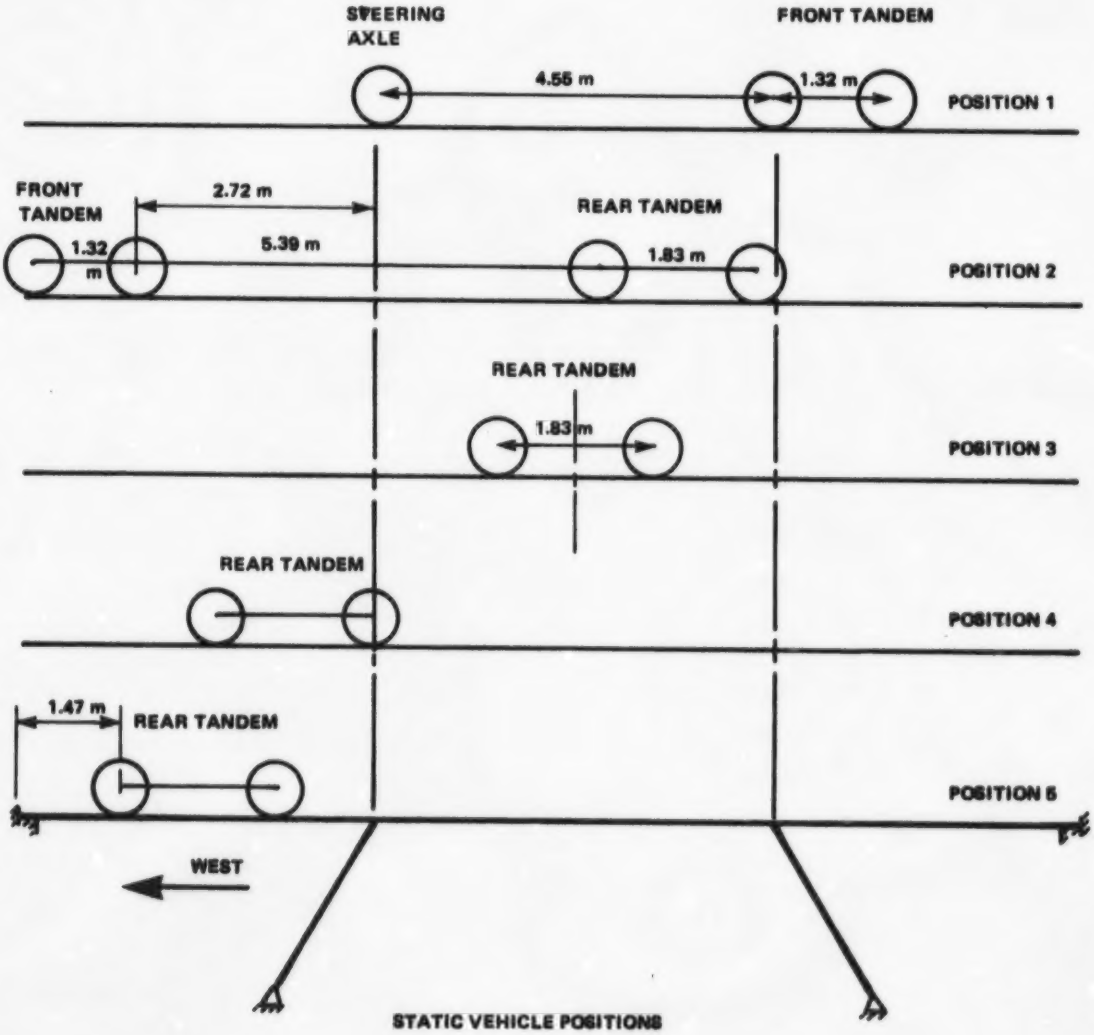


Figure 26/ Additional Instrumentation

Figure 27/ Demountable Strain Gauge



VEHICLE LANE POSITIONS



STATIC VEHICLE POSITIONS

Figure 28/ Vehicle Placement

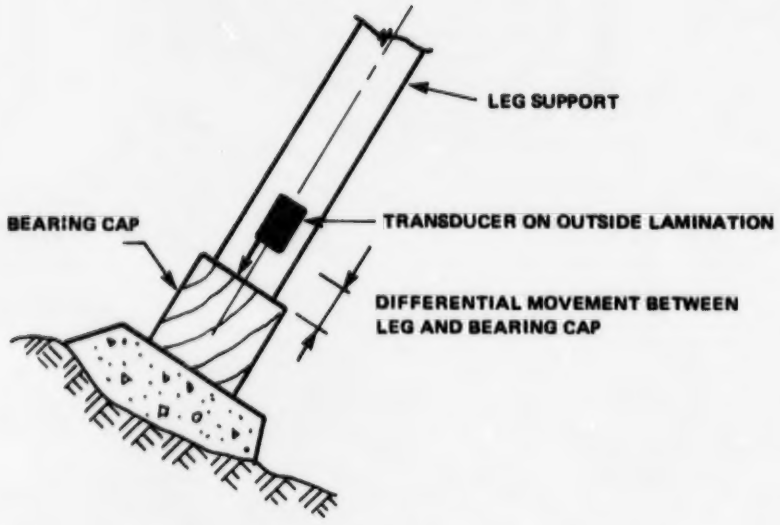


Figure 29/ Monitoring Leg Movement

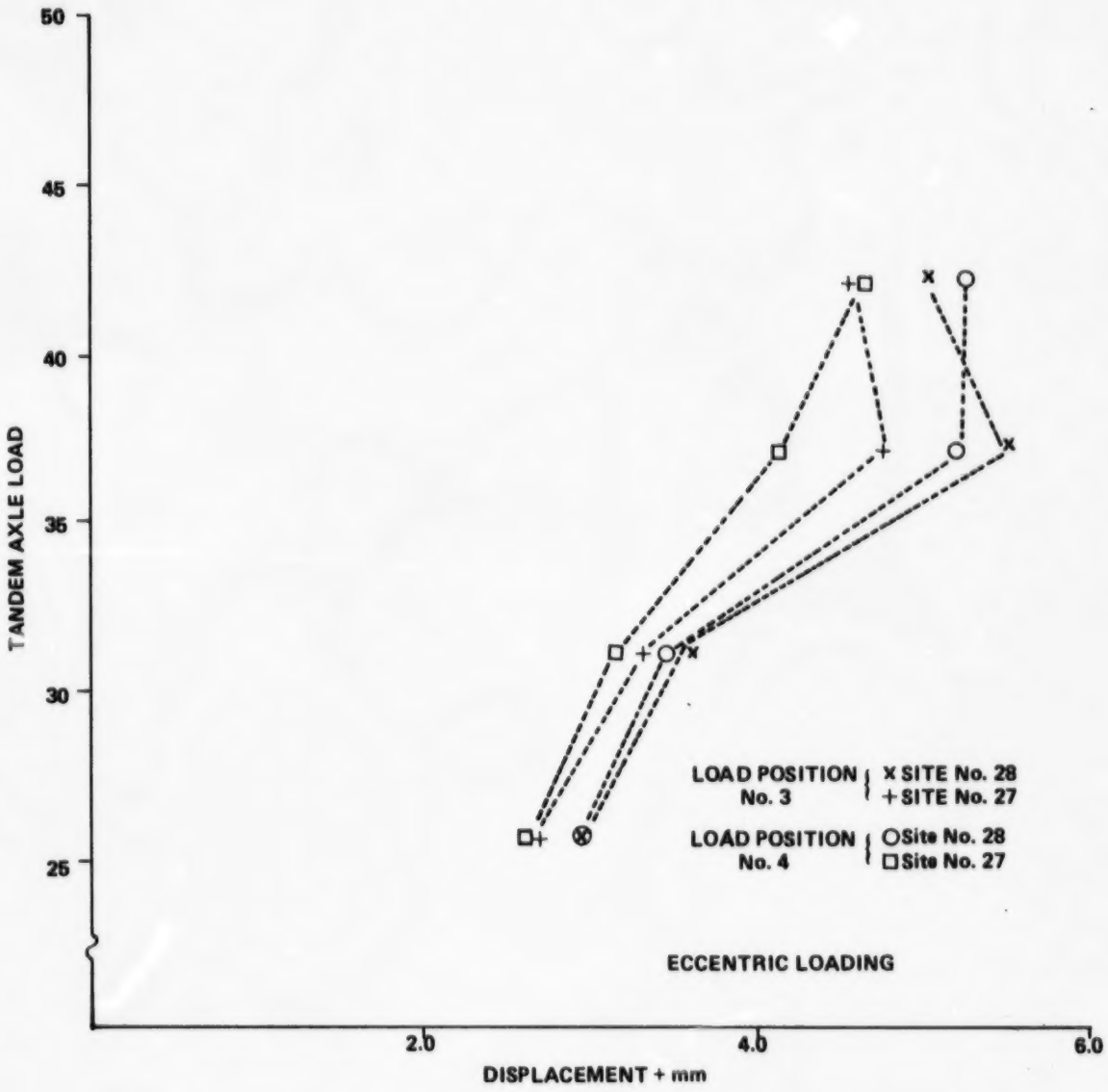


Figure 30/ Vehicle Load versus Vertical Displacement at Leg Support

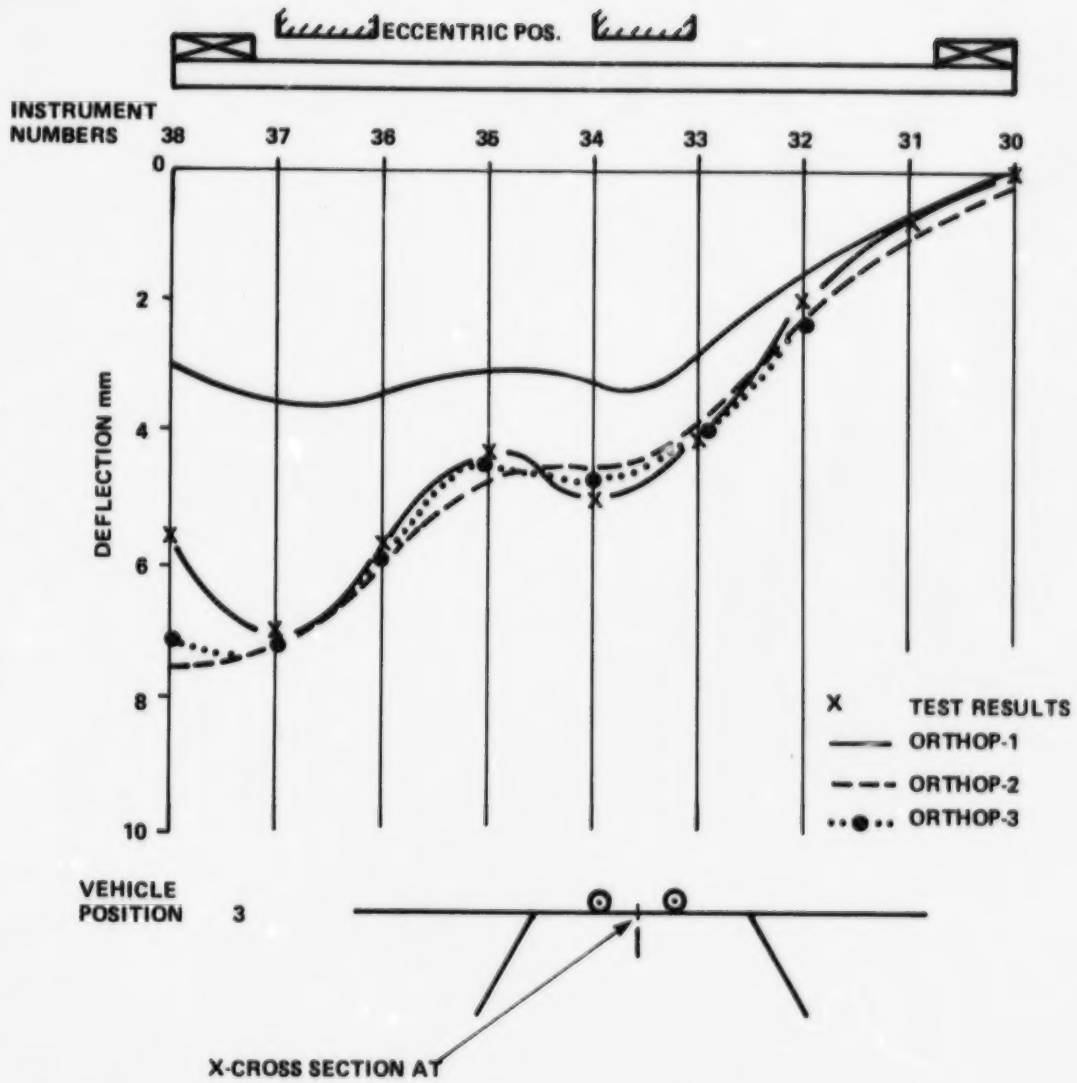


Figure 31/ Vertical Displacements, Centre Span, Eccentric Loading

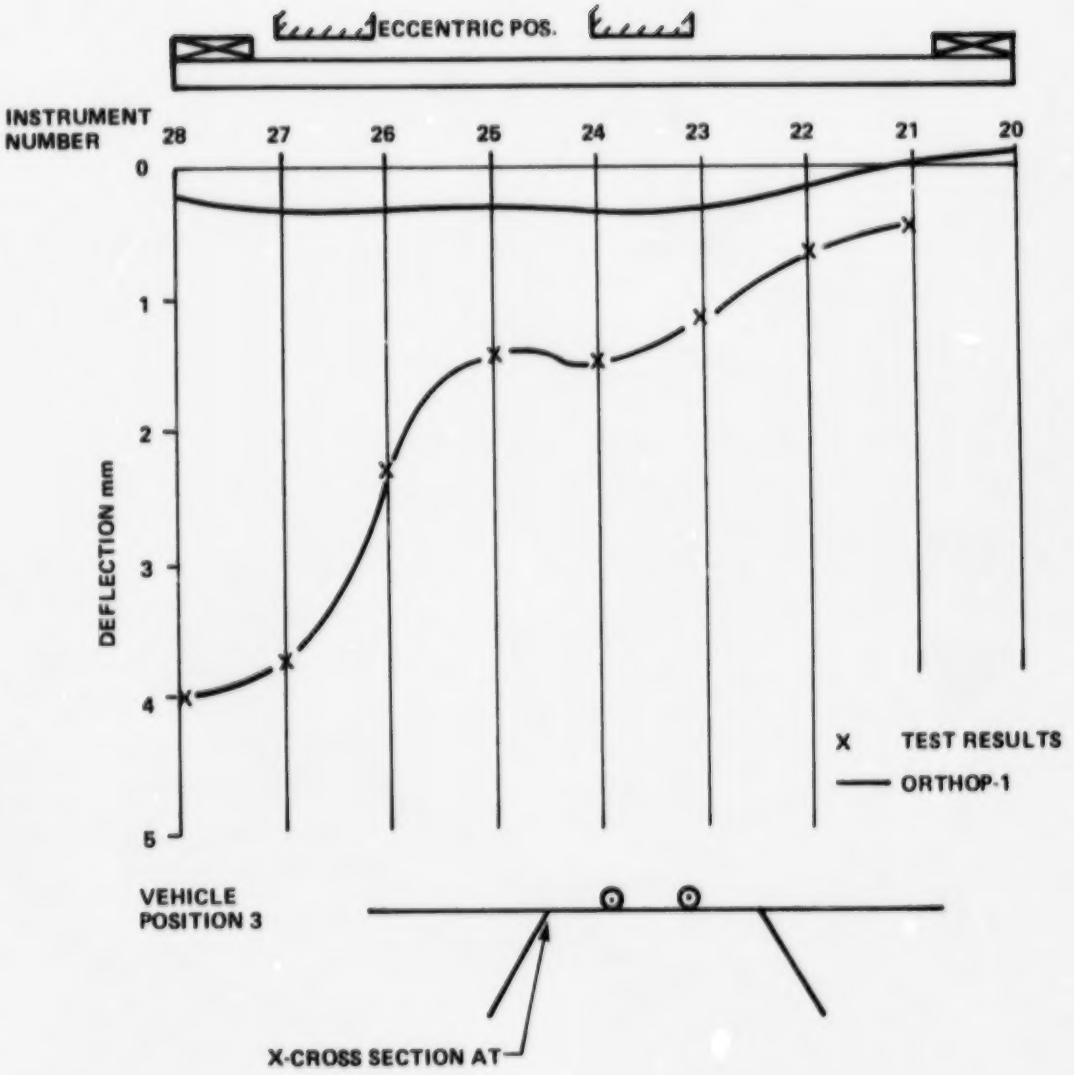


Figure 32/ Vertical Displacements, West Leg Support, Eccentric Loading

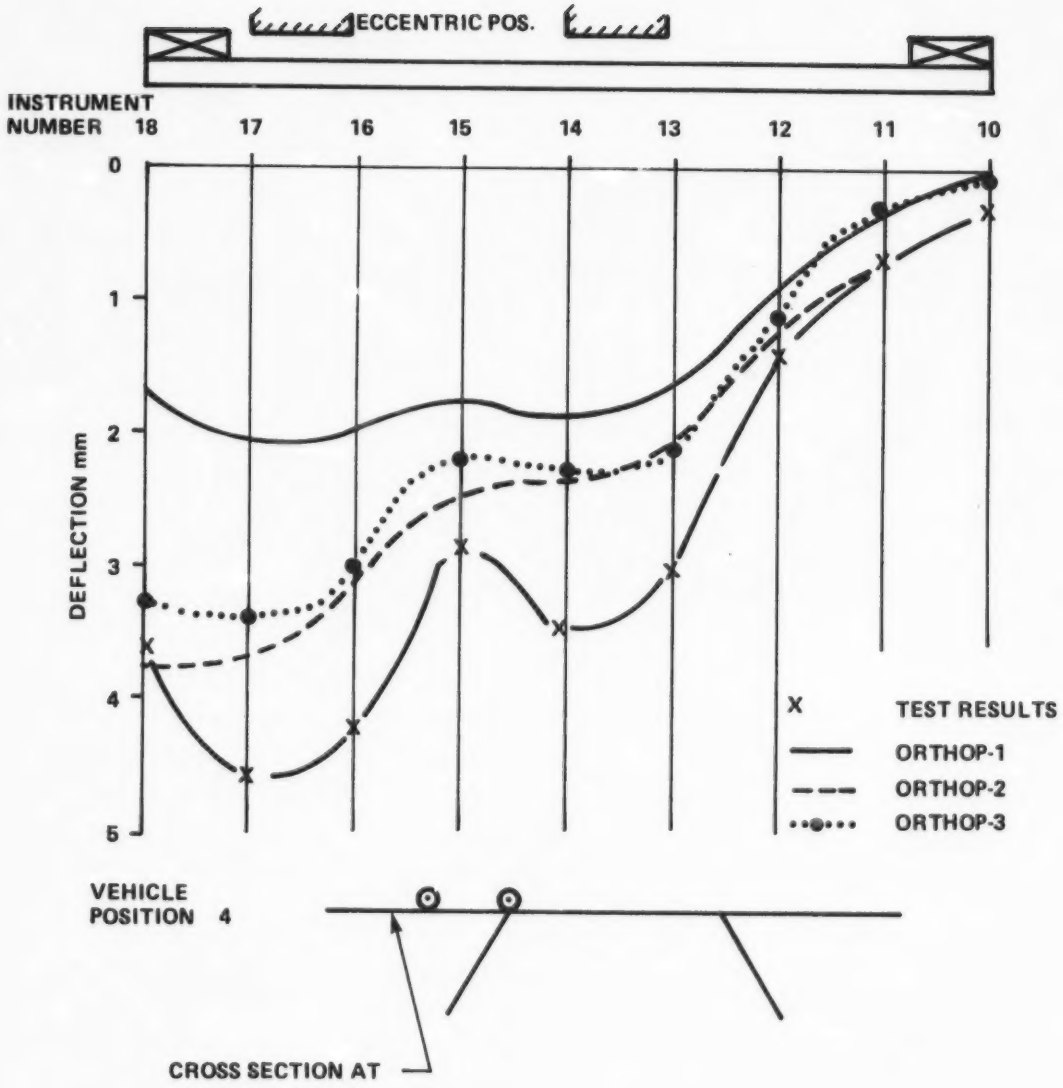


Figure 33/ Vertical Displacement, West Span, Eccentric Loading

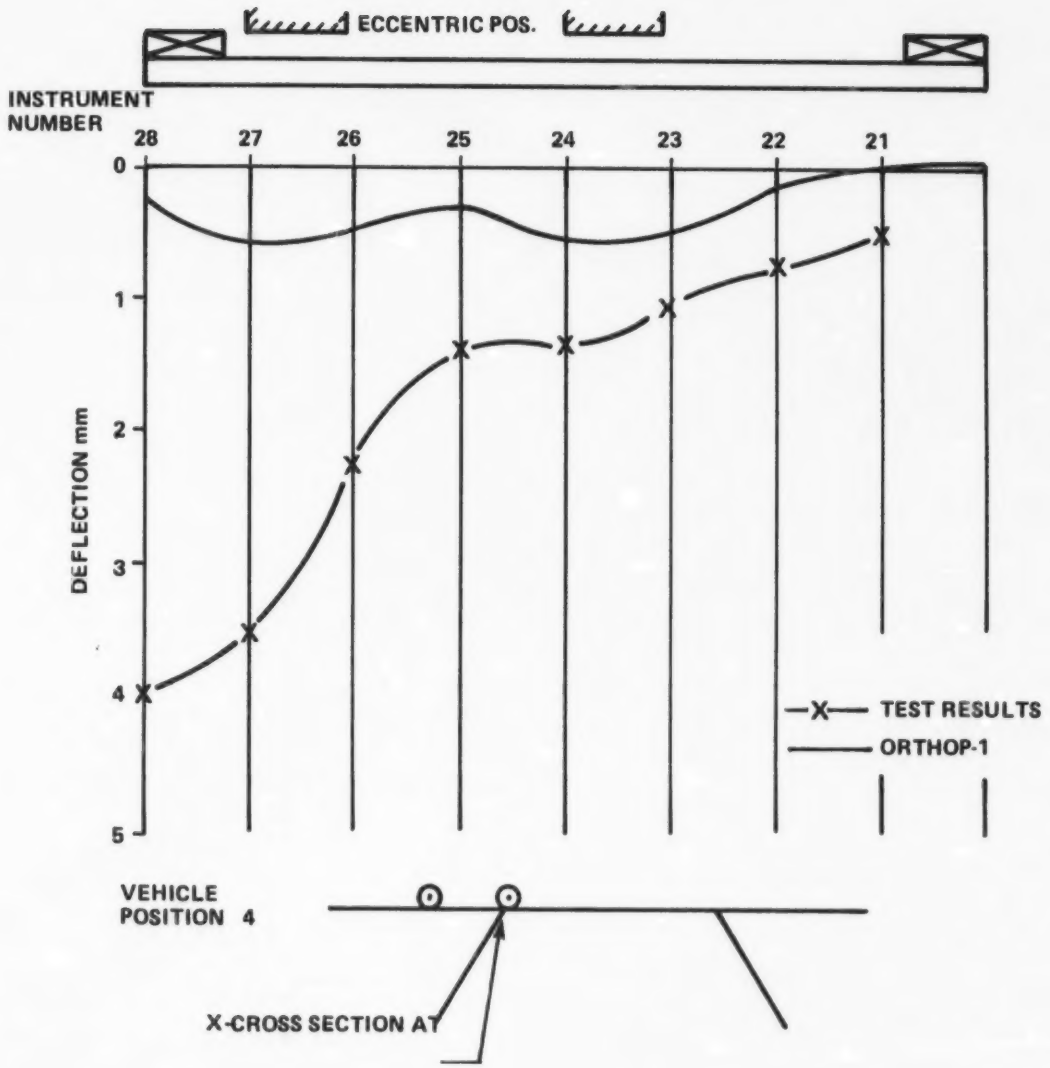


Figure 34/ Vertical Displacements, West Leg Support, Eccentric Loading

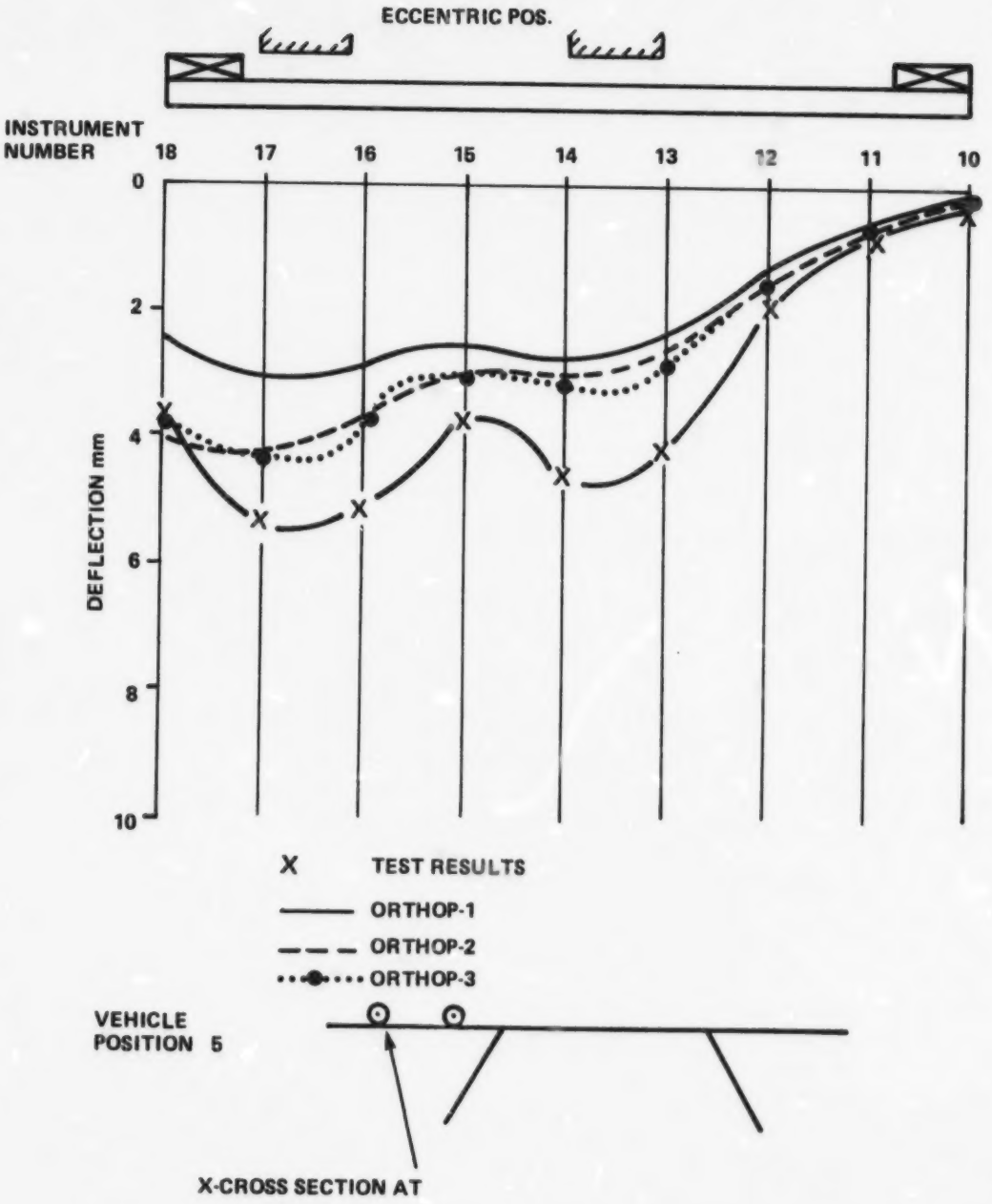


Figure 35/ Vertical Displacements, West Span, Eccentric Loading

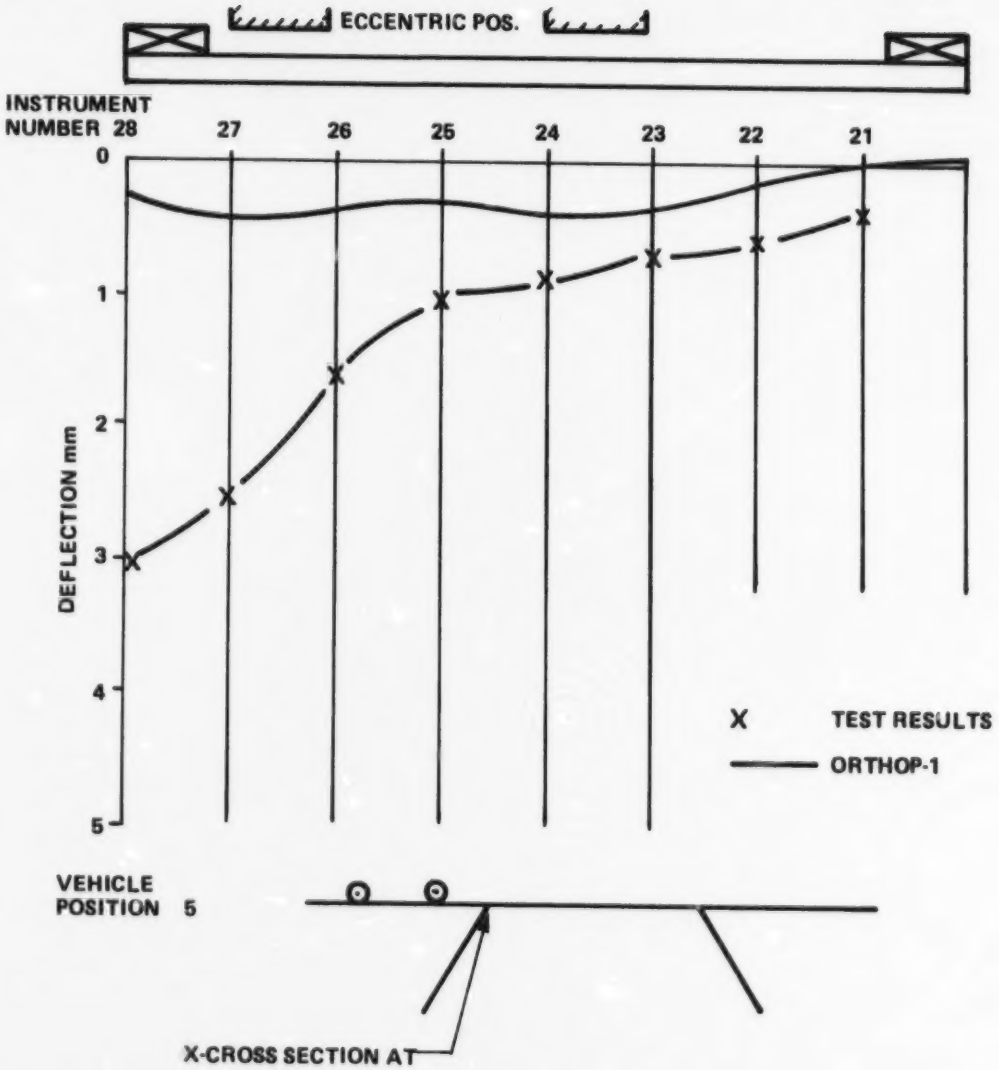


Figure 36/ Vertical Displacements, West Leg Support, Eccentric Loading

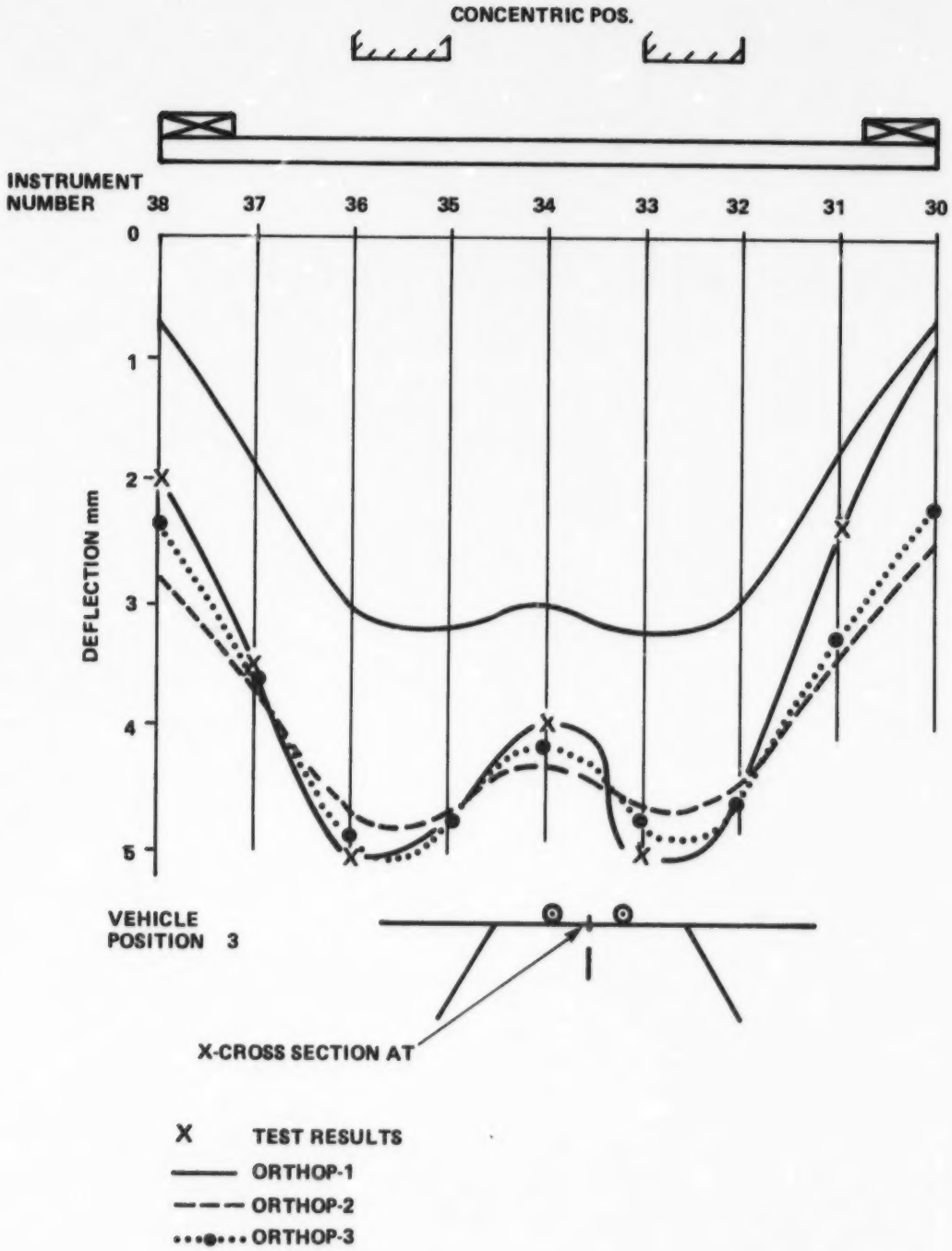


Figure 37/ Vertical Displacements, Centre Span, Concentric Loading

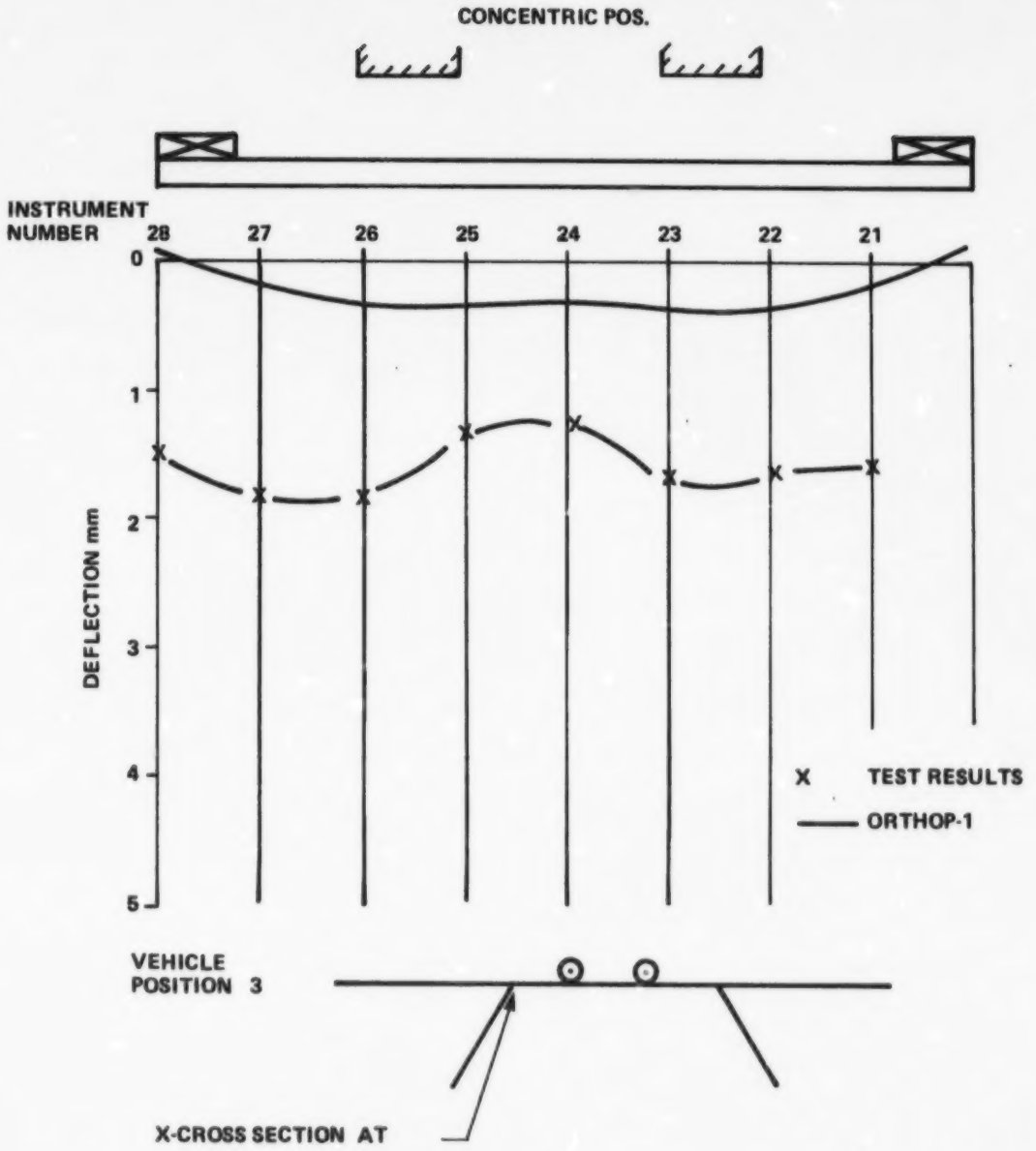


Figure 38/ Vertical Displacements, West Leg Support, Concentric Loading

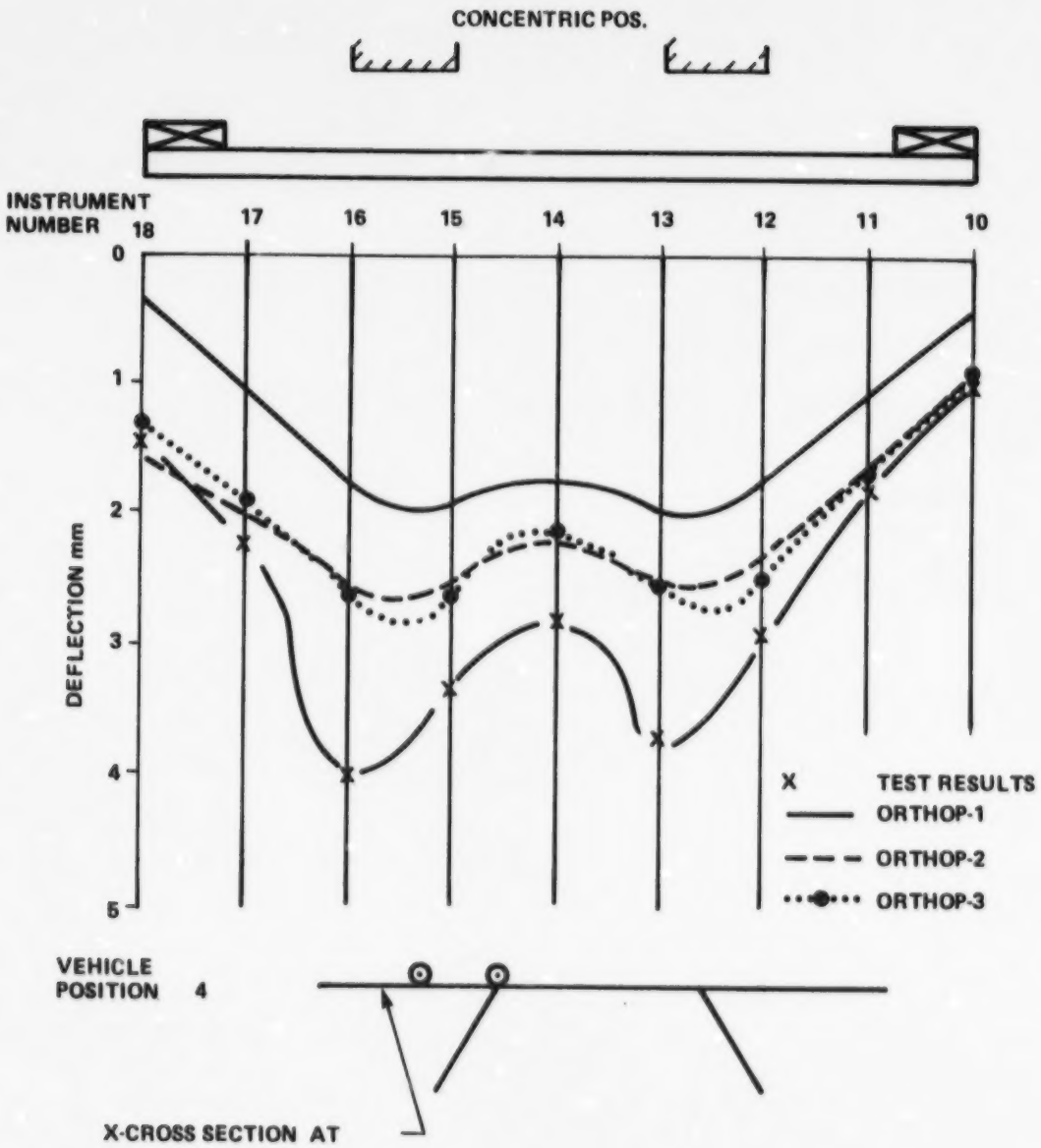


Figure 39/ Vertical Displacements, West Span, Concentric Loading

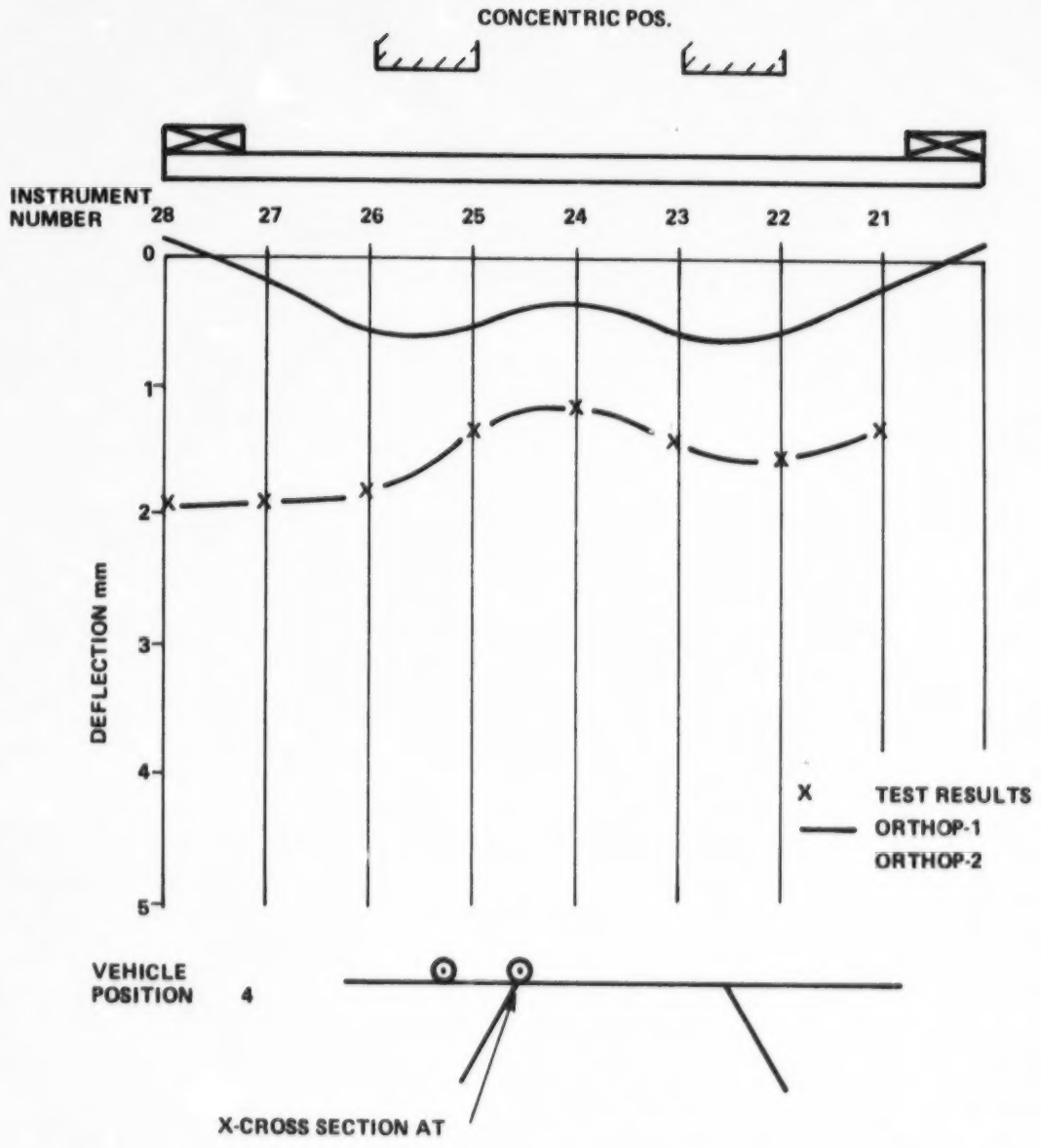


Figure 40/ Vertical Displacements, West Leg Support, Concentric Loading

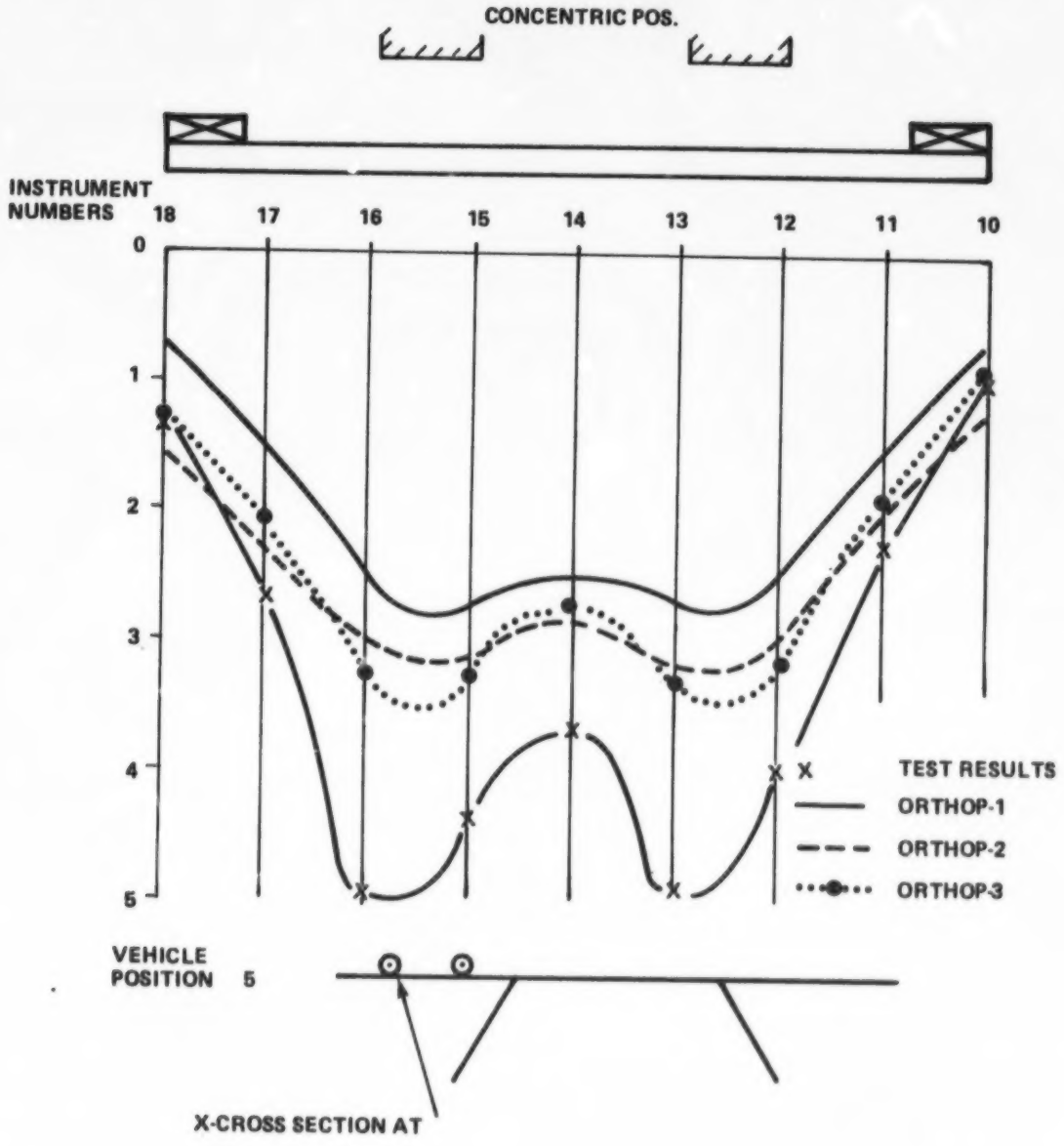


Figure 41/ Vertical Displacements, West Span, Concentric Loading

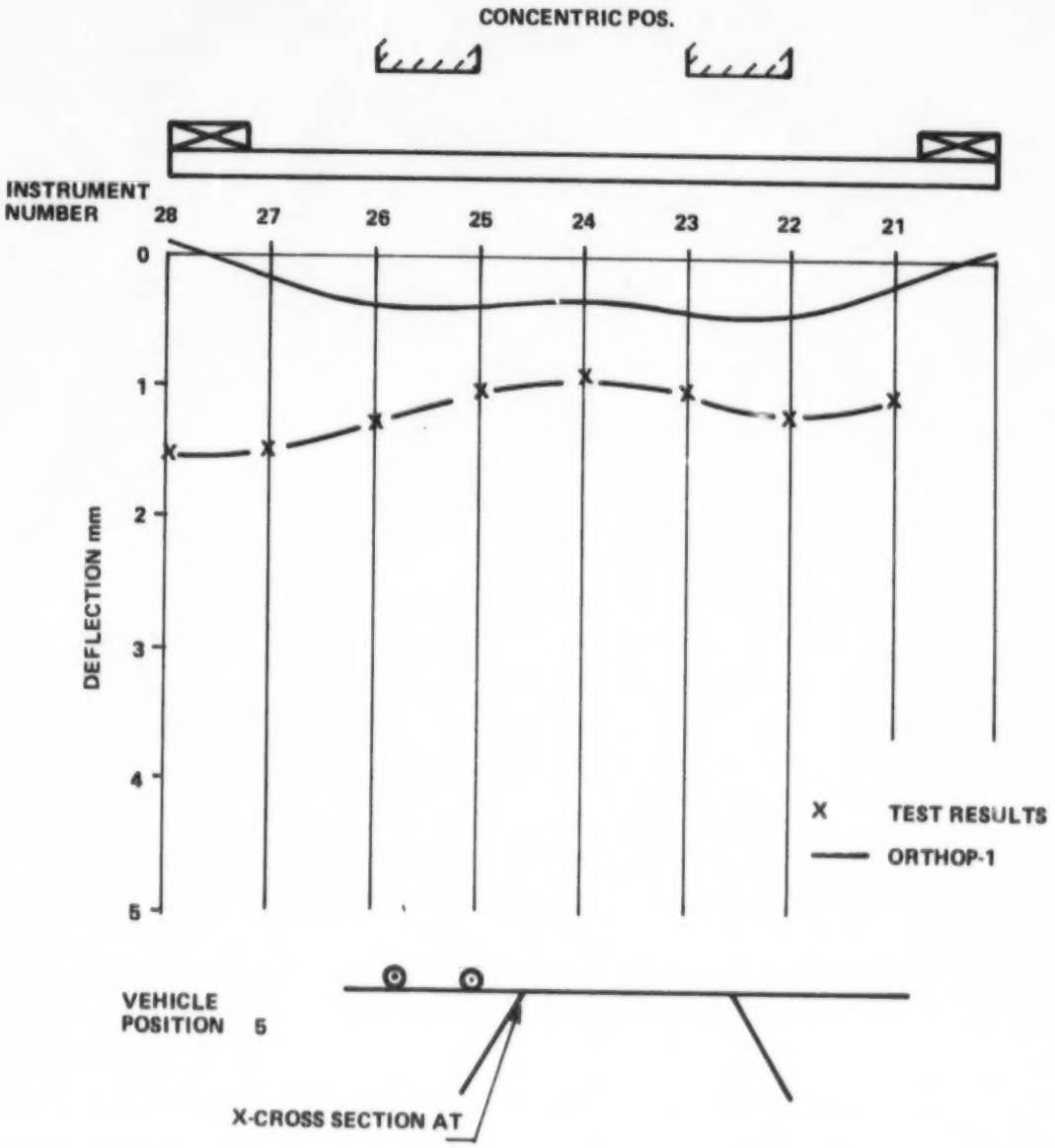


Figure 42/ Vertical Displacements, West Leg Support Concentric Loading

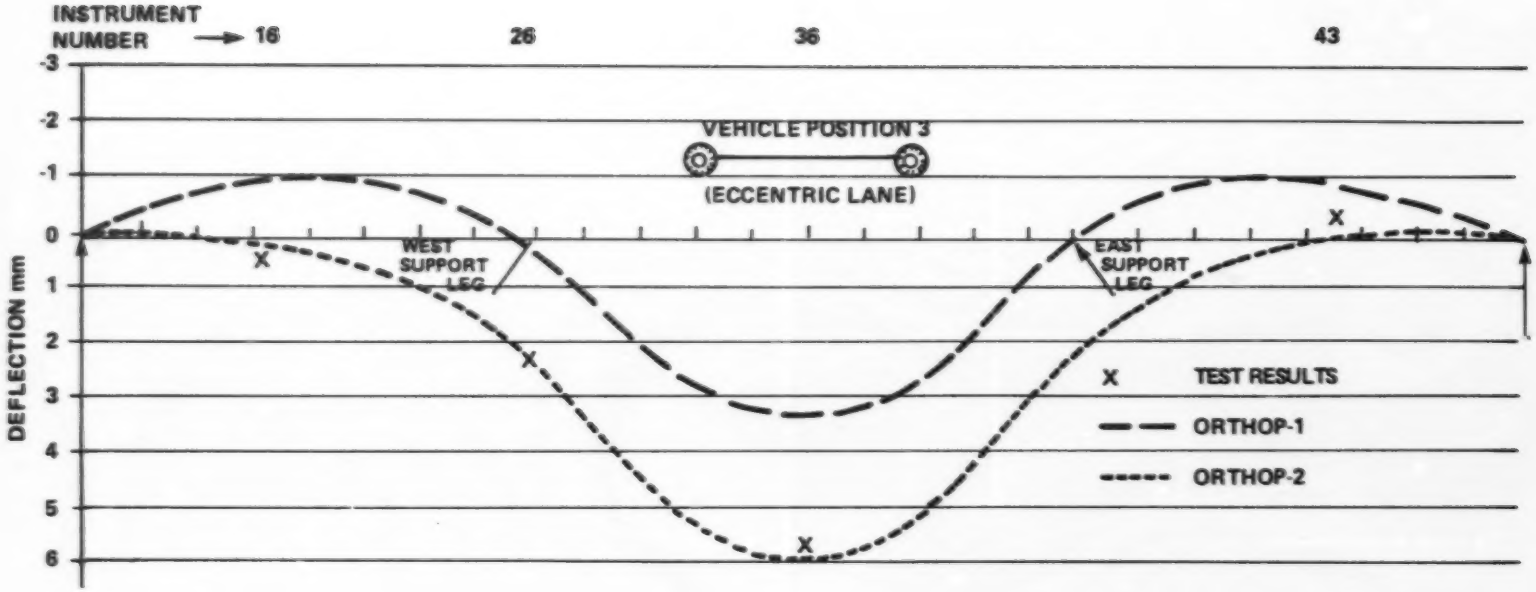


Figure 43/ Vertical Displacements, Vehicle Position 3, Eccentric Loading

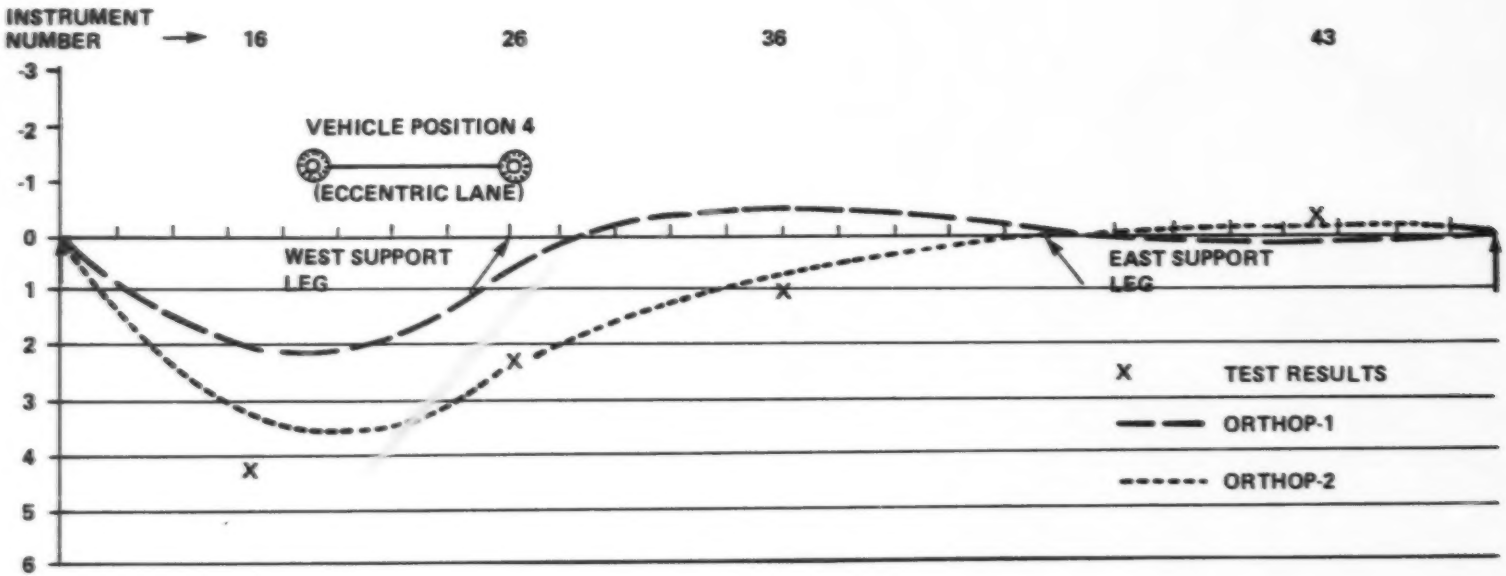


Figure 44/ Vertical Dispalcements, Vehicle Position 4, Eccentric Loading

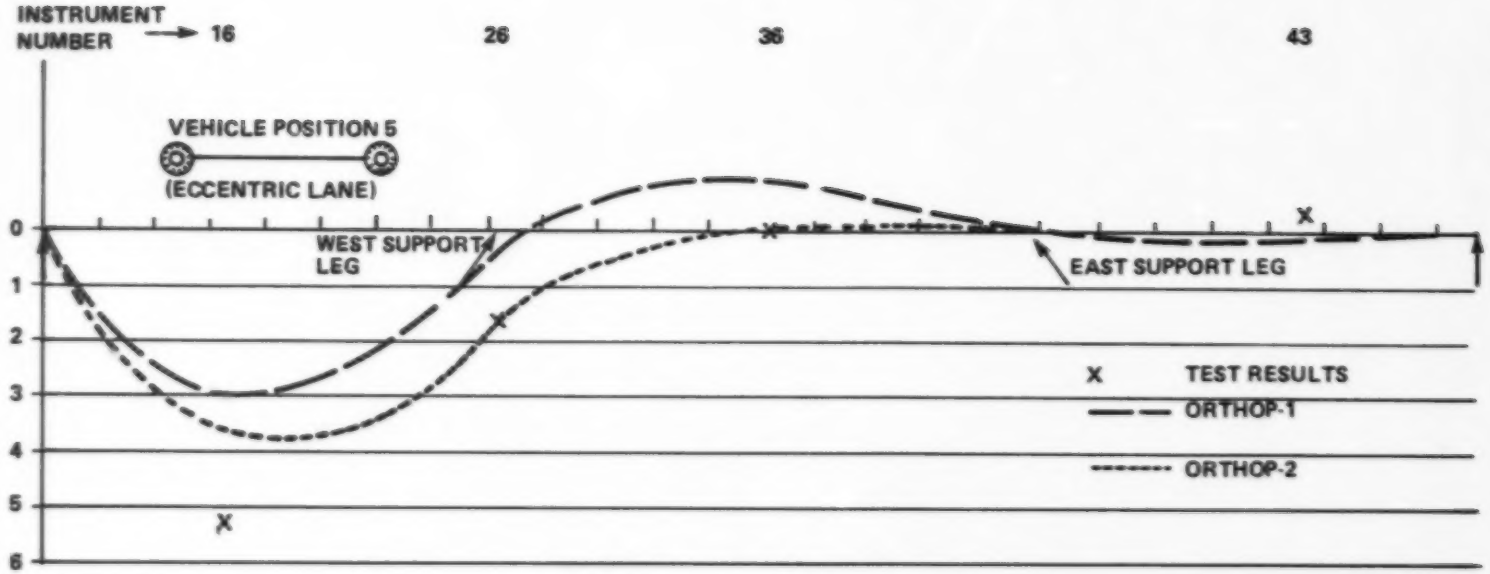


Figure 45/ Vertical Displacements, Vehicle Position 5, Eccentric Loading

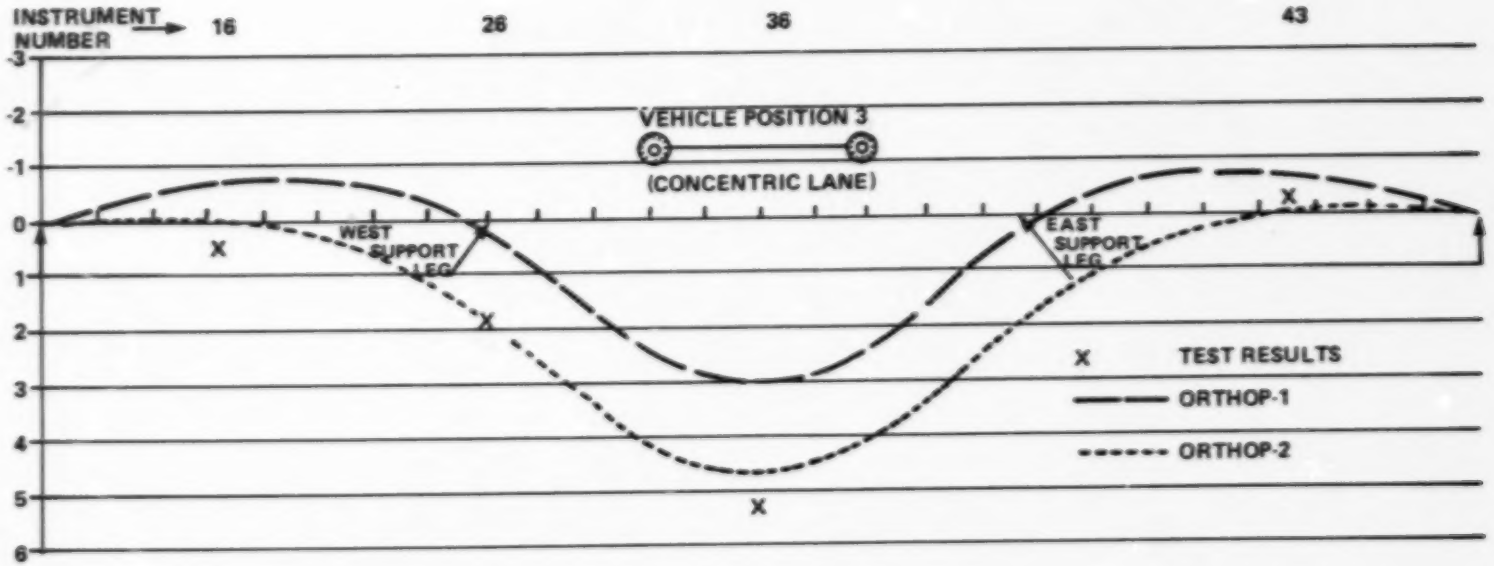


Figure 46/ Vertical Displacements, Vehicle Position 3, Concentric Loading

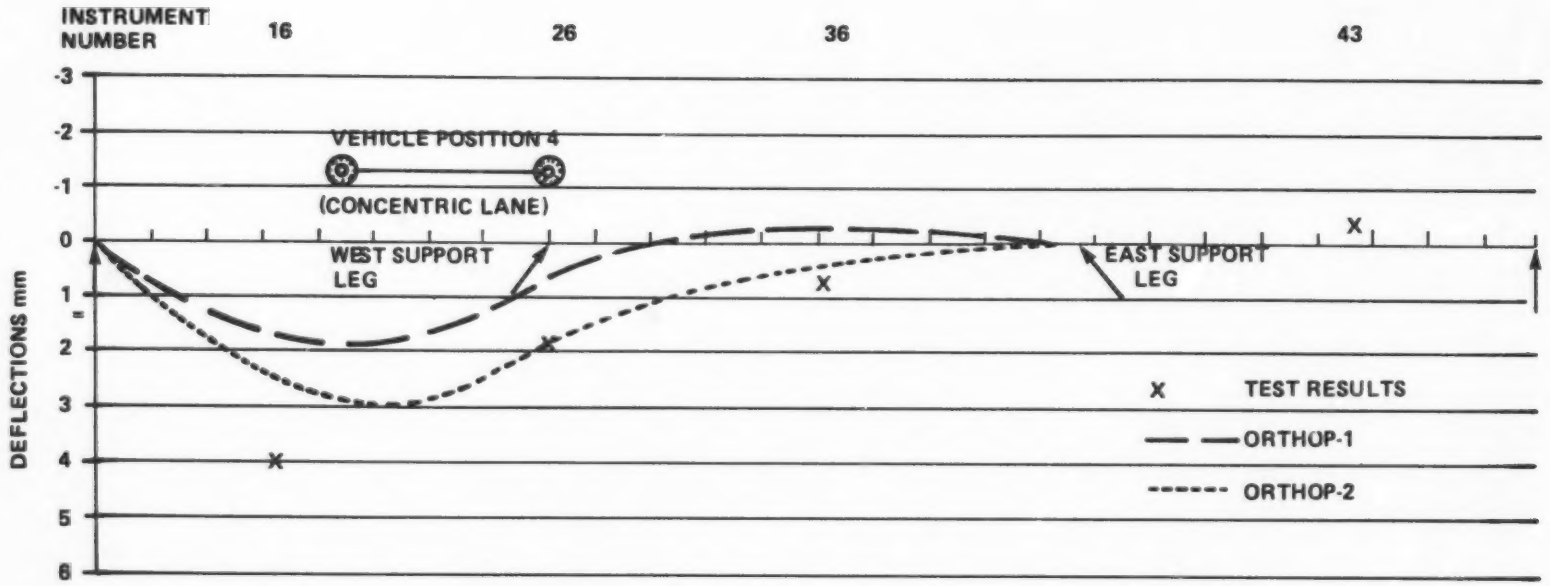


Figure 47/ Vertical Displacements, Vehicle Position 4, Concentric Loading

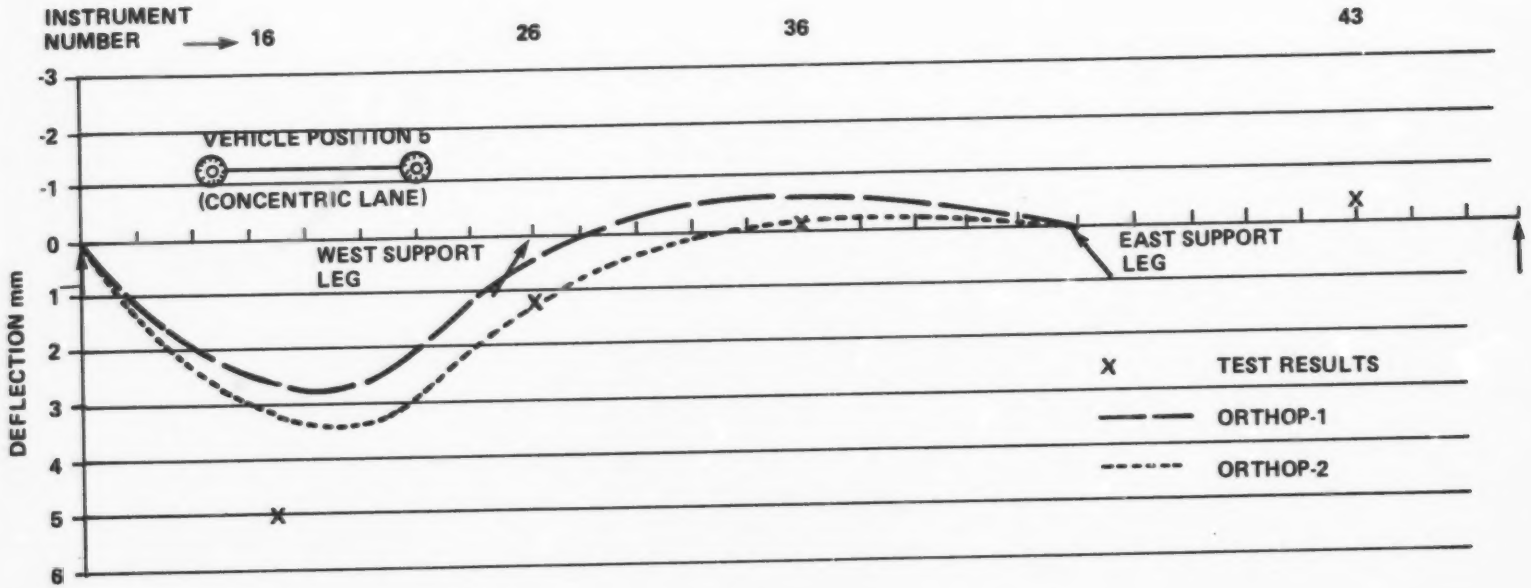


Figure 48/ Vertical Displacements, Vehicle Position 5, Concentric Loading

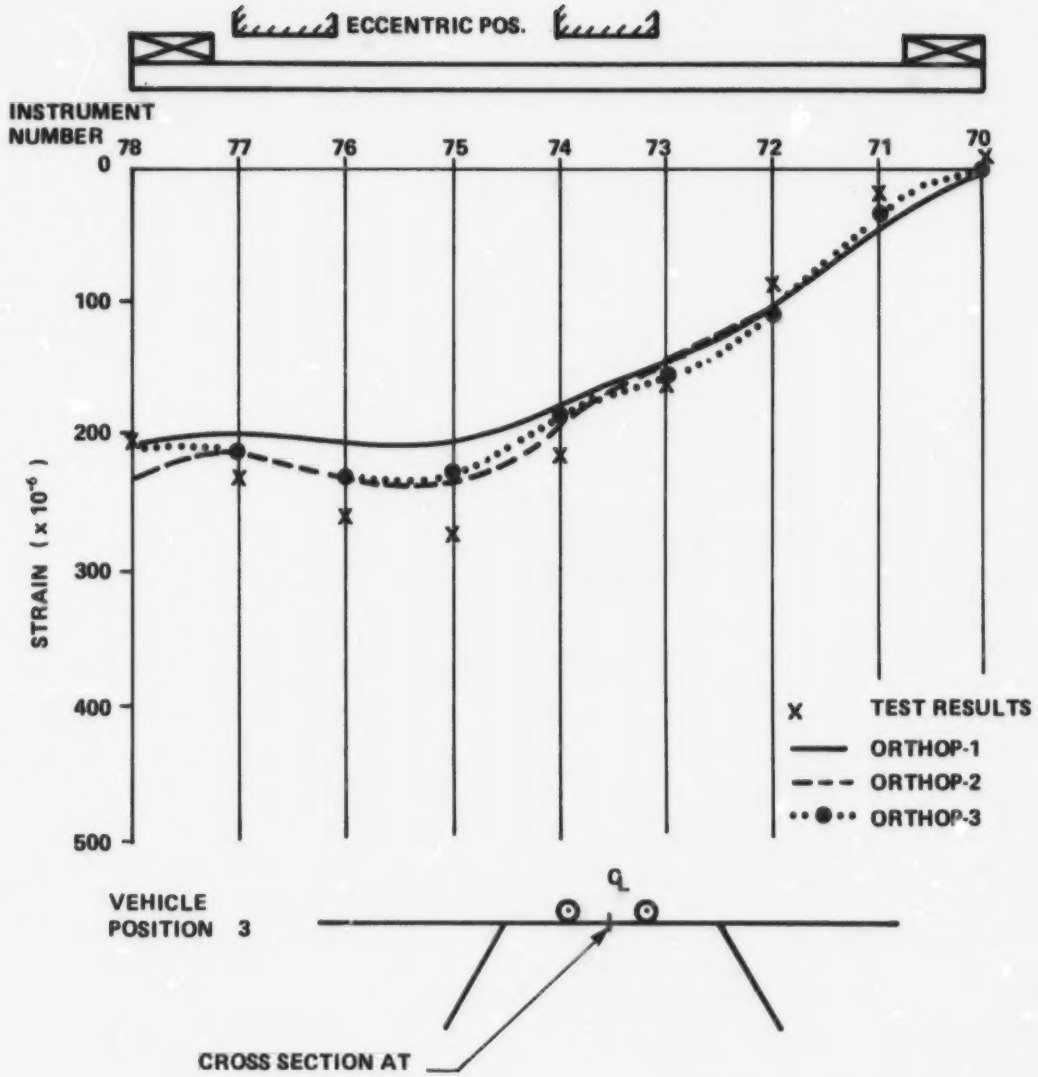


Figure 49/ Flexural Strains, Vehicle Position 3, Eccentric Loading

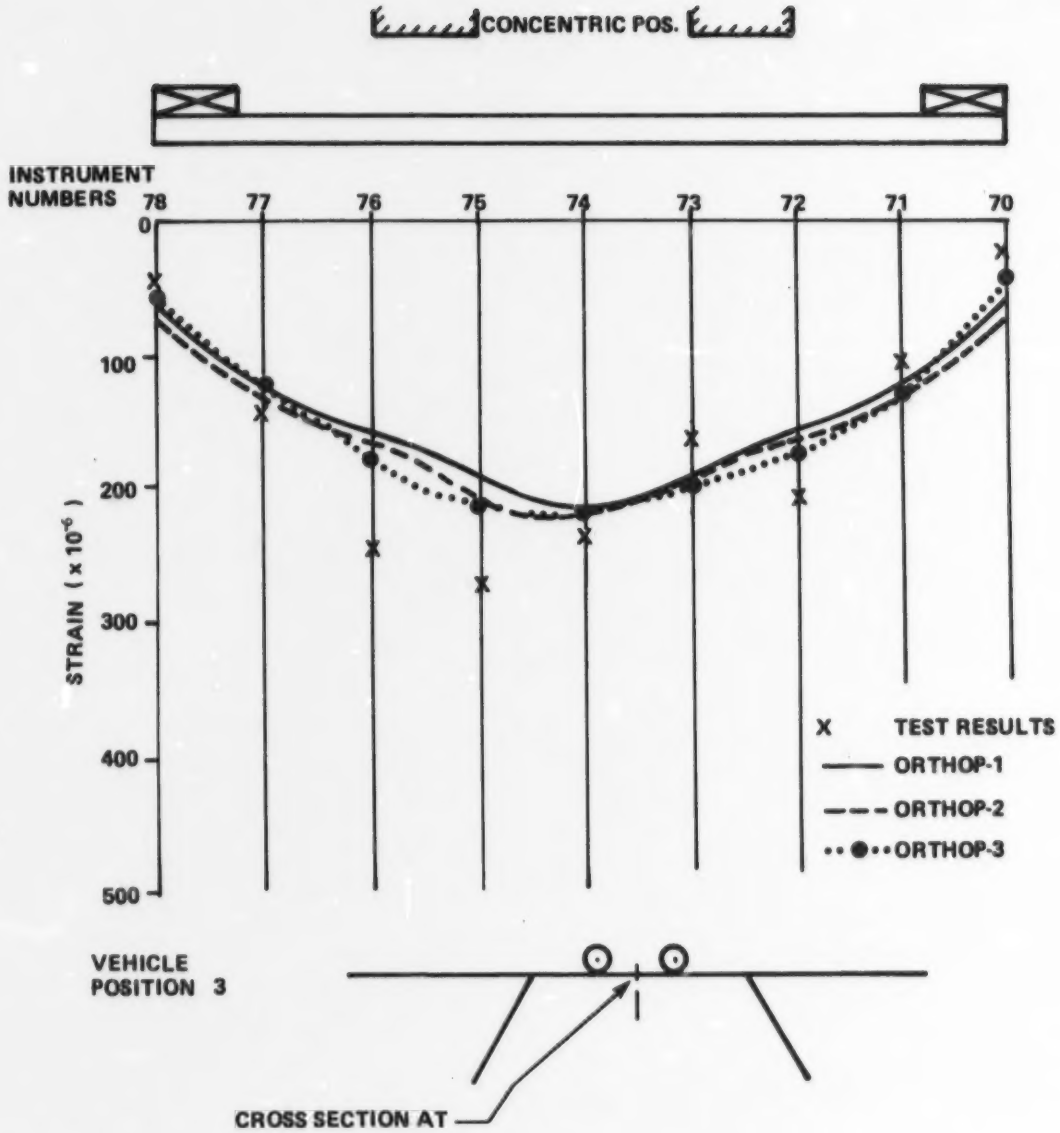


Figure 50/ Flexural Strains, Vehicle Position 3, Concentric Loading